FOR GOD AND MY COUNTRY

THE REPUBLIC OF UGANDA
MINISTRY OF WATER AND ENVIRONMENT

WATER SUPPLY DESIGN MANUAL
SECOND EDITION

AMENDMENT HISTORY

DATE	CLAUSE	AMENDMENT

ABBREVIATIONS

ABBREVIATION/ACRONYM	MEANING
AC	Asbestos Cement
ADD	Average day demand
APHA	American Public Health Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
AWWA	American Water Works Association
BPS	Booster pump station
CRS	Capacity-related storage
CSV	Cycle stop valve
CT	Chlorine concentration x time
DBP	Disinfection by-product
DDP	District Development Plan
DE	Diatomaceous earth
DI	Ductile Iron
DOH	United States Department of Health
DS	Dead storage
DSL	Distribution system leakage
EAW	Equivalent Annual Worth
EDB	Ethylene Dibromide
EIA	Environment Impact Assessment
EMP	Environmental Monitoring Plan
EOCC	Economic Opportunity Cost of Capital
EPA	Environmental Protection Agency
EPS	Extended-period simulation
ERA	Electricity Regulatory Authority
ERU	Equivalent residential unit
ES	Equalizing storage
ETV	Environmental testing verification
EU	European Union
FGD	Focus Group Discussions
FIRR	Financial Internal Rate of Return
Fps	Feet per second
FSS	Fire suppression storage
GAC	Granular activated carbon
GS	Galvanized Steel
HC	Health Centre

ABBREVIATION/ACRONYM	MEANING
HDPE	High Density Polyethylene
IFC	International Finance Corporation
IMF	International Monetary Fund
IRR	Internal Rate of Return
ISO	International Standards Organization
LC	Local Council
MARR	Minimum Acceptable Rate of Return
MPa	Mega Pascal
MWE	Ministry of Water and Environment
NEA	United States National Environment Act
NEMA	National Environment Management Authority
NFA	National Forestry Authority
NPV	Net Present Value
Pa	Pascal
PAP	Project Affected Person
PE	Person Equivalent
PLRA	Precise Location of Resistivity Anomaly
SEIA	Social and Environment Impact Assessment
ToR	Terms of Reference
UAIA	Uganda Association for Impact Assessment
UNBS	Uganda National Bureau of Standards
uPVC	Un-Plasticized Polyvinyl Chloride
US	Uganda Standard
USEPA	U.S. Environmental Protection Agency
UTM	Universal Transverse Mercator
UWA	Uganda Wildlife Authority
WB	World Bank
WHO	World Health Organization
WPCF	Water Pollution Control Federation
WSP	Water system plan
WSP	Water Supply Projects
WTP	Water Treatment Plant
WUE	Water use efficiency

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Chapter 1

INTRODUCTION

1.1 General Introduction

The Ministry of Water and Environment published the First Edition of the Water Supply Design Manual in the year 2000. This Manual has been extremely useful in meeting the needs of those engaged in the planning and design of water supply systems. Over time, however, new technologies in the design of water supply systems have evolved; a lot of experience has been gained, and new issues such as climate change have emerged that have necessitated the revision of this Manual.

1.2 Purpose of the Water Supply Design Manual

The Water Supply Design Manual is to ensure that the planning and design of water supplies is formalized, follows set procedures, and similar processes and procedures are implemented.

1.3 Structure of the Manual

The Second Edition of the Water Supply Design Manual follows that of the first edition. This is to enable those who are used to the first edition to find their way through the Second Edition. There is however, a significant expansion of content in most of the chapters and the introduction of new chapters covering social and environmental assessments; financial and economic analysis.

The chapters and their contents are arranged as follows:

Chapter 1: Introduction - background and policy and legal framework for water supply in Uganda.

Chapter 2: Water Demand - methods of estimating water demand.

Chapter 3: Water Sources - types of water sources.

Chapter 4: Water Intakes - types of intakes associated with water sources.

Chapter 5: Water Quality - national and relevant international water quality issues.

Chapter 6: Water Treatment - commonly used treatment options in Uganda.

Chapter 7: Water Transmission and Distribution - pipeline design for water supply.

Chapter 8: Water Pumping - the guidelines for sizing pumps for raw and treated water and the various sources of power for the pumps.

Chapter 9: Treated Water Storage - information on the types and capacities of water storage tanks in water supply.

Chapter 10: Environmental and Social Impact Assessments - environmental and social Impact assessment captured by a baseline study.

Chapter 11: Cost Estimates - the various methods by which cost estimates for water supply may be calculated.

Chapter 12: Financial and Economic Analysis - the possibilities for financial and economic analysis of alternatives to guide the Designer in making choices for development.

Chapter 13: Design Reports - the outlines of the main reports expected from the Designer of the water supplies.

1.4 Policy, Legal and Institutional Framework for Water Supply in Uganda

1.4.1 Policy Framework

Water supply in Uganda is governed by the National Water Policy. The Policy created the Water Policy Committee [WPC] to assist and advise the Minister of Water and Environment and to promote Inter-Ministerial and inter-sectoral coordination over a wide range of water resources management and development. The WPC provides an avenue for promoting IWRM at national level and guiding the strategic management and development of water resources of the country. The WPC also coordinates the preparation of national water quality standards; and mediations and undertakes conflict resolution between national authorities on water resources matters [MWE, 2011].

Important policies that impact the sector include:

i) Uganda Water Action Plan (1995);
ii) The National Environment Policy (1994);
iii) National Health Policy and Health Sector Strategic Plan (1999); and
iv) National Gender Policy (1997).

1.4.2 Strategic Framework

Important strategies include:

i) Water Supply and Sanitation Sector Investment Plans and Allocation Principles (SIP 2008-2035) (2009);
ii) Rural Water and Sanitation Operation Plan 2002-2007 (OP5);
iii) Memorandum of Understanding between the Ministry of Water and Environment (MWE), Ministry of Health (MoH) and the Ministry of Education and Sports (MoES) (Dec 2001);
iv) National Framework for Operation and Maintenance of Rural Water Supplies (2004);
v) Rural Growth Centres Strategy (2005);
vi) Strategy for Water and Environmental Sanitation Emergency Response in Uganda (2004);
vii) Steps in Implementation of Water and Sanitation Software Activities (MWE/DWD in 2005);
viii) Strategy for Mainstreaming HIV/AIDS (2004);
ix) Long Term Strategy for Water Supply and Sanitation Services in Small Towns (MWE/DWD, 2003);
x) The Improved Sanitation and Hygiene Strategy (ISH), (2006);
xi) Sanitation Mobilization Steps (MWE/DWD, 2004); and
xii) Planning Guidelines for Hygiene Promotion and Education (MoH/EHD, 2005).

1.4.3 Regulatory Framework

The Ministry of Water and Environment through its Directorates of Water Development (DWD) and that of Water Resources Management (DWRM) is responsible for the regulation of water supplies in Uganda. The Ministry, through the National Environment Management Authority and the Wetlands Department is responsible for the regulation of the environment and the wetlands from which water supplies are made [Water Act, Cap 152].

During implementation of water supply projects, the important acts and regulations to be consulted are [ULRC, 2012]:

i) The Constitution of the Republic of Uganda;
ii) The Water Act, Cap 152, and several regulations, such as those on water resources and water quality made under the Act;
iii) The Local Governments Act, Cap 243,;
iv) National Water and Sewerage Corporation Act, Cap 317;

v) The Public Health Act, Cap 281;
vi) The National Environmental Act, Cap 153, and several regulations such as EIA Regulations, Statutory Instrument No. 13, 1998, the National Environment (Standards for Discharge of Effluent into Water or on Land) Regulations, S.I. No 5/1999, the National Environment (Hilly And Mountainous Area Management) Regulations, 2000, etc. made under the Act;
vii) The Uganda Wildlife Act, Cap 200;
viii) The Land Act, Cap 227; and
ix) The Town and Country Planning Act, Cap 246.

Regulation of the persons who may be involved in the design of water supplies is covered by:

i) Engineers Registration Act, Cap 271;
ii) Surveyors Registration Act, Cap 275;
iii) Architects Registration Act, Cap 269; and
iv) National Environment Act, Cap 153

The key professionals involved in the design of water supplies should be registered accordingly and should be licensed to practice their respective professions in Uganda.

1.4.4 Institutional Framework

The institutional framework of the water and environment sector is captured in Figure 1-1.

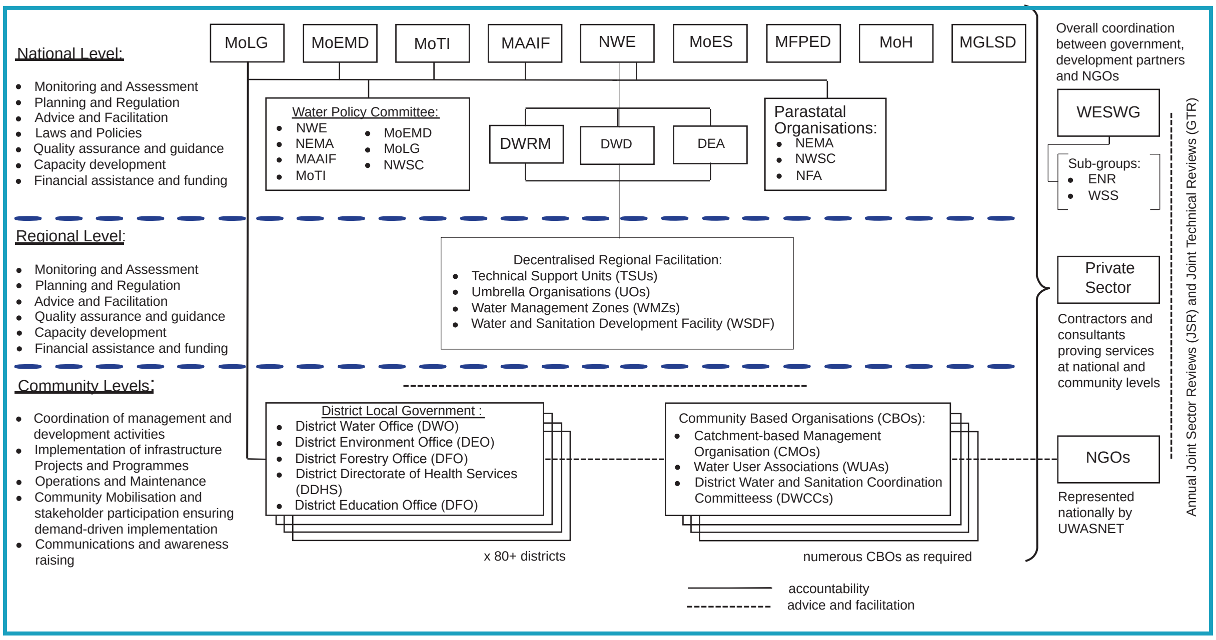


Figure 1-1: Institutional Framework for the Water and Environment Sector, 2011.
Source: MWE, 2011: Water and Sanitation Sector Performance Report, 2011.

1.5 Basic Design Principles of Water Supply in Uganda

A water supply project should be designed on the basis of the following principles:

i) Capacity to satisfy demand at the projected service levels over the design horizon;
ii) Provision of adequate safe and sanitary (potable) water;
iii) Wise, effective, efficient and environmentally friendly use of the water resource;
iv) Safe and sound operation and maintenance of the water facilities;
v) The system and its operations to conform to the Water Act and any regulations and guidelines related it;

vi) Enhancement of the quality of living standards of the consumers; and
vii) Appropriate and cost effective technology relevant to the beneficiaries of the system.

1.6 History of Water Supply in Uganda

The first water treatment plants in Uganda were constructed in Entebbe and in Gaba, in Kampala, by the Colonial Administration of the Uganda Protectorate. Gaba Water Works is the largest of the water treatment plants in Uganda, serving the Greater Kampala Metropolitan Area which stretches from Gaba to Kawempe Division to the North, Mukono Municipality to the East and to Wakiso District in the East

The Government supplies water to the urban centres of Kampala, Entebbe and Jinja through the National Water and Sewerage Corporation which was created under the NWSC Act. In addition, NWSC supplies water to 20 urban centres – the largest of the urban centres in Uganda. The small towns and rural growth centres are supplied water by the Directorate of Water Development through the large towns' water and sanitation boards, districts or through the private sector.

1.7 Technology Choices

The common technologies used in water supply in Uganda are:

i) Rainwater harvesting;
ii) Springs;
iii) Shallow (dug) wells;
iv) Deep boreholes;
v) Rock catchment;
vi) Dams;
vii) Gravity fed systems;
viii) Pumped water supplies; and
ix) Bulk water transfer systems.

Apart from bulk water supplies, the rest of the technologies have been widely used in Uganda for water supplies. The Ministry of Water and Environment has prepared strategies to move large quantities of water from areas where there is excess supply to those areas that are water stressed.

This Manual does recognize the existence of systems that can be developed for bulk water supplies such as:

i) The Rwenzori Mountains that can supply large parts of Kasese and surrounding areas;
ii) The Mount Elgon Mountains that can supply large parts of Karamoja; Sebei; Teso and Bugisu with water; and
iii) The Lake Kyoga system which can supply areas of Nakasongola, Lango and Acholi with water.

These systems require large scale social and environmental impact assessments before they can be implemented and will certainly require the indulgence of the neighbouring states since these water systems are trans-boundary.

1.8 Instances of Failure of Water Supplies in Uganda

Failure of large water supplies is rare in Uganda. Boreholes, springs and gravity flow sources and distribution systems have failed on a small scale. For the large sources, the failures or inadequacies are corrected on emergency basis since the adage in Uganda is that 'water is life' – total failure is simply unacceptable. Common among the failures and inadequacies of water supplies are the following:

i) Failure of the water treatment system to provide water of acceptable quality;
ii) Failure of the pumping, storage and distribution system to supply adequate water supply to the consumers;
iii) Failure of boreholes to provide adequate quantity and/or quality of water;
iv) Design failures where the new systems have failed to perform due to poor system design;
v) Destruction of water supplies by storm water, earth movements, lightning strikes or other natural catastrophes;
vi) Drying up of the main source of raw water;
vii) Abandonment of water sources due to pollution; and
viii) Abandonment of sources due to environmental considerations such as gazetting of the source area as a protected zone.

Examples of serious water system failure or inadequacy include:

i) Pallisa Town Water Supply which has consistently produced poor water quality;
ii) Bushenyi/Ishaka Municipal Water Supply System which has failed to produce water of acceptable aesthetic quality;
iii) Several gravity flow schemes that have been destroyed by flooding in the Elgon and Ruwenzori mountains;
iv) Numerous boreholes that have dried up or have got polluted with faecal matters or with salt water intrusion along the shores of Lake Victoria and Lake Kyoga; and
v) Numerous springs and wells that were poorly sited during the wet season only to dry up during the dry season or get polluted with surface water during extreme wet seasons.

1.9 Comparative Analysis of Some Water Supplies in Uganda

1.9.1 Performance Rating of NWSC Towns

National Water and Sewerage Corporation is given an annual Performance Contract to operate and manage water supplies in the major towns of Uganda. The Corporation operates in twenty three towns namely Kampala (including Kajjansi and Nansana), Jinja/Njeru, Entebbe, Tororo, Mbale, Masaka, Mbarara, Gulu, Lira, Lugazi, Fort Portal, Kasese, Kabale, Arua, Bushenyi/Ishaka, Soroti, Mukono, Malaba, Iganga, Mubende, Hoima, Masindi and Kaberamaido. Mukono water services are managed by Kampala Area; Malaba is under Tororo Area [NWSC, 2010]. Through its operations, it provides water and sewerage services to about 2.3 million people out of the targeted 3.1 million. Around 72% of NWSC water output amounting to 72 million cu.m is supplied to Kampala and its environs, making this the most important for their operations. NWSC has shown a remarkable improvement in its operations over the last 10 years, leading to improved financial performance.

NWSC keeps and publishes comprehensive sets of data for its operations. These data show differing levels of compliance with national and other standards as well as performance data for all the NWSC towns. The majority of the town water supplies are from surface water, which requires conventional treatment, thus the aesthetic quality parameters are important indicators of system performance. Capacity utilization is a two-faced indicator. For Kampala, with a capacity utilization of 90%, the emergency capacity is minimal and indicates that the treatment capacity should be increased.

On the other hand, low capacity utilization such as in Soroti (29%) indicates over-design or broken down systems. The system designer should avoid either extreme.

While the majority of samples have adequate removal of E-Coli, there is a problem with the aesthetic quality of some of the towns. Aesthetic quality may be represented by colour, turbidity and TSS, since they give the consumers the feeling that the water quality is not acceptable. All the samples were found wanting in one or more of the parameters, with the greatest problem being in Ishaka/Bushenyi Municipality.

Designers should be careful to improve on the aesthetic quality of water without greatly increasing the cost of treatment. This may be achieved by a combination of treatment methods and inclusion of slow sand filtration wherever possible.

According to the performance criteria, the top eight best performing towns in the large urban areas: (1) Fort Portal, (2) Kasese, (3) Entebbe, (4) Kabale, (5) Arua, (6) Kampala, (7) Gulu and (8) Lugazi, based on the percentages compliance with set standards and capacity utilization as a percentage of installed capacity. Kampala, the largest of the urban centers by far, has almost full capacity utilization, which boosts its scores but it performs less in the other parameters. These parameters have inbuilt design elements (efficiency of treatment systems) and operational elements. The global unit consumption figures are given in Table 1- 2.

Table 1-1: Percentage Compliance for Treated Water Quality in NWSC Towns, 2010

Area	Colour	Turbidity	TSS	Residual Chlorine	E-Coli	Capacity use	Billing efficiency	Total
Fort Portal	96	100	93	94	100	89	79.6	652
Kasese	100	100	93	93	100	80	82.7	649
Entebbe	100	100	99	100	99	54	89.2	641
Kabale	100	100	91	92	100	65	91.0	639
Arua	92	98	92	93	100	67	90.9	633
Kampala	90	100	98	93	100	90	60.8	632
Gulu	92	100	90	94	100	71	84.3	631
Lugazi	100	100	95	94	100	51	77.3	617
Mbale	99	100	94	99	100	32	92.2	616
Masindi	94	100	90	91	100	48	89.2	612
Jinja	99	100	94	93	99	50	74.1	609
Soroti	94	99	92	97	100	29	85.5	597
Hoima	94	100	90	89	100	34	86.1	593
Masaka	90	99	89	90	100	58	66.7	593
Mbarara	80	95	80	81	100	64	88.6	589
Mubende	88	99	90	87	96	37	87.1	584
Lira	86	100	76	88	100	41	87.0	578
Tororo	80	94	78	90	97	40	90.3	569
Bushenyi	5	60	66	48	100	53	80.8	413
Average	97	100	94	94	100	63	82.8	630

TSS: Total Suspended Solids
Source: NWSC, 2010 and 2012 (excludes Iganga for which some of the data was missing).

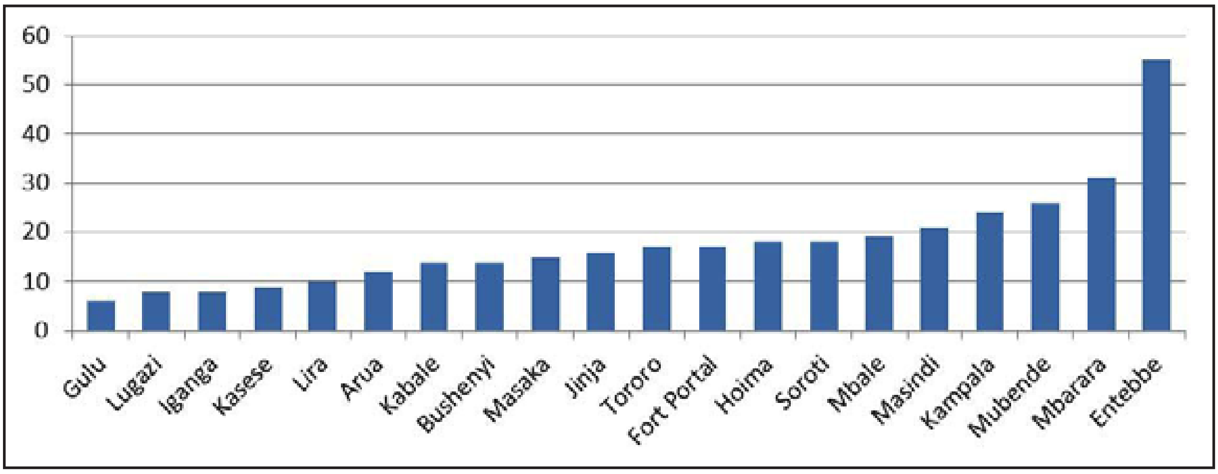


Figure 1-2 Consumption Volume (m³/year) per Connection in NWSC Towns [NWSC, 2010]
Source: NWSC, 2010.

Table 1-2: Per Capita Water Sales Per Customer Category For NWSC Towns.

Consumer category	No. of connections	Total water sold (m³/year)	Water sales per connection (m³/d)	Water sales per connection (l/d)
Public Stand posts	7,748	2,257,335	0.8	798.20
Domestic	194,848	22,996,603	0.3	323.35
Institutions / Government	6,686	9,781,786	4.0	4,008.29
Industrial / Commercial	36,977	11,992,093	0.9	888.53
Total	246,259	47,027,817	0.5	523.20

Source: NWSC, 2012, p. 30.

1.9.2 Comparative Performance of Small Towns

The data available for the top five Small Towns in each WSDF is shown in Table 1-3. The data is does not have the same parameters as for the NWSC towns; however, comparisons can be made among the towns. The majority of the towns have water supplies based on groundwater or gravity fed systems and many of them have the addition of disinfectant as the only treatment since groundwater is generally potable and aesthetic quality is taken as acceptable (which is the case in most cases).

Table 1-3 Comparison of five towns from each WSDF

Towns	Billing Efficiency%	% of Active Connections	Collection Efficiency %	% of O&M Funded By Revenue	Total	Region
Semuto	80	90	96	257	523	WSDF-C
Kalisizo	89	90	107	233	518	WSDF-C
Lukaya	88	98	103	219	507	WSDF-C
Luwero	80	93	101	225	499	WSDF-C
Bukomansimbi	88	99	90	179	456	WSDF-C
Kumi	91	65	103	228	487	WSDF-E
Kamuli	77	83	84	239	483	WSDF-E
Sironko	85	91	76	202	455	WSDF-E
Budadiri	85	61	100	208	454	WSDF-E
Masafu	92	87	59	192	431	WSDF-E
Moyo	68	86	108	237	499	WSDF-N
Pakele	85	78	77	229	470	WSDF-N
Nebbi	77	90	77	225	469	WSDF-N
Koboko	92	95	97	174	459	WSDF-N
Amolatar	96	100	152	88	436	WSDF-N
Ntungamo	100	50	99	404	653	WSDF-S
Kabwohe - Itendero	63	95	84	329	570	WSDF-S
Rukungiri	86	83	49	298	515	WSDF-S
Kihihi	90	92	87	188	456	WSDF-S
Kisoro	65	98	108	162	434	WSDF-S

Source: DWD, 2011: ST KEY DATA FY10-11 Q12 2011-02-28

A high percentage of O&M costs that are funded by revenue indicate that the system is operating very efficiently with minimal O&M costs compared to the revenues. It is desirable that the revenues exceed the O&M costs to ensure system sustainability though this could also indicate that the system is relatively new. Older systems tend to have higher O&M costs.

Again, the operational parameters are overriding but the designs would have contributed to make the systems easier to operate.

1.9.3 Small Towns and Rural Growth Centres Under DWD

For the small towns and rural growth centers, the figures (generated from data supplied by MWE) show a remarkably similar situation. The majority of the towns consume in the range 30 – 60m^3/year (2.5 – 5m^3/month), which compares well with the consumption in the larger towns under NWSC.

The average consumption figure for the largely domestic consumption lies between 20 l/c/d (0.6 m^3 per month for stand pipes) and a high of 200 l/c/d (6 m^3 per month for the high income house connections). The above consumption figures from NWSC bear these figures out. These figures are therefore retained in this edition of the WS Manual.

The consumption for public stand posts under conditions similar to those of the NWSC towns should be estimated at 1.0 m^3 per day where the number of users is indeterminate. For smaller towns and RGCs, a range of consumption may be adopted depending on local circumstances.

A number of criteria have been set for the performance of water supply systems in Uganda. In accordance with the Performance Agreement signed by the Minister in charge of Water and the district and urban authorities, the performance obligations of the Water Authority include: to own assets; to provide services; to charge for services provided and to comply with laws of Uganda, among others.

1.10 Problems with Existing Water Supply Programs

1.10.1 Small Towns and Rural Growth Centres under DWD

The small towns under the Rural Water and Sanitation Department cover all the towns and rural areas that are not under NWSC. More than 80 towns with water supplies are managed by private sector firms some of which are under the Association of Private Water Operators. The Private Operators are procured by the Ministry and local governments in accordance with the provisions of the Water Act and the Public Procurement and Disposal of Public Assets Act and its Regulations (PPDA).

A major problem with the POs is the failure to make viable businesses since the towns are individually tendered out, making it uneconomical to manage some of the small, remote towns. Consolidation of the towns is being planned by the Ministry.

Major maintenance of the systems, such as replacement of pumps or other costly parts, usually reverts to the Ministry of Water since they are outside the means of the individual firms, which can lead to delays in restoring supplies to the affected towns. Attempts are being made to encourage the POs to contribute to an assets recovery fund from which funds for these major repairs can be sought.

Sales of water in some of the towns are not as high as projected by the POs at the time of tendering since many of the towns have alternative sources of water. These alternative sources are getting increasingly polluted however due to increased human habitation of the catchments; however, until the cost of water is reduced significantly, affordability remains a problem and some consumers tend to use unsafe sources.

The Ministry is working to establish an assets register for all the small towns in order to have orderly management and valuation of the installed assets. Lack of an assets register impedes the Ministry in planning long term maintenance of the systems.

1.10.2 NWSC Towns

NWSC has shown a remarkable improvement in its operations over the last 10 years, leading to improved financial performance. However, according to the Annual Report [NWSC, 2010, p. 23], the main challenges are non-revenue water, arrears and old networks and expansion of existing infrastructure.

In a number of towns, there are reports of shortage of water. In Bushenyi, interventions were done in 2010 to expand and rehabilitate the system [NWSC, 2010, p. 26].

The sewerage systems in all the towns and Kampala City cover only the core towns and are inadequate. They were planned and constructed for a fraction of the present populations. Investment costs for new sewerage works are quite high and beyond the means of most towns.

1.11 Evaluation of Some Guideline Used in the Water Supply Manual

1.11.1 Introduction

The Designer is often faced with the decision of what demand, non-revenue water or other parameter to include in the design. Data collection in the water sector has appreciably improved over the last few years, thus enabling comparative analysis to be done. Using data from NWSC covering over 20 towns and MWE covering over 85 towns, we have selected demand and non-revenue water for more detailed analysis.

1.11.2 Demand Estimates

The Water Supply Manual [MWE, 2000], demand for towns is categorised according to domestic, institutional and commercial categories. In any town, there is a combination of all the three categories, each according to the level of industrialization of the town. Mainly residential towns are expected to have similar consumption patterns. A typical connection uses approx. 10 – 20 m^3 per month, assuming that the majority of connections represented are domestic and institutional (mainly schools).

1.11.3 Estimates of Non-Revenue Water

Non-revenue water is water lost through leakage, theft of water, pipe cleaning requirements, social requirements (mainly fire fighting) and failure to collect invoiced amounts. The Designer concerned with determining the demand of a town will consider leakage, pipe cleaning requirements and social requirements (mainly fire fighting) as the main water losses that should be included in the assessment of demand. Theft and non-settlement of bills are operational losses.

NWSC operates the largest and oldest piped water network in the country, with Kampala, Jinja and Entebbe being the lead towns in this regard. It is no surprise therefore that these towns have high non-revenue water figures. For more than half the towns under NWSC, the losses vary from 20% to a high of 40% in Kampala. On the other hand, the small towns graph shows a slightly better picture.

The larger of the towns, such as Pallisa, Ngora, Kapchorwa, Busolwe and Nagongera have high losses, much more than the recommended design value of 10 – 15%. The newer systems, such as Kasambira, Katakwi, Amolatar and Kumi have low losses, ranging from 3 – 10%. This compares well with the design guides given in the Manual.

The conclusion is therefore that non-revenue water, and for the designer, unaccounted for water, should be assessed at 20 – 25% as recommended, with still higher figures for some of the rehabilitation areas, which can rise to 40% of the water supplied.

Significantly, the Designer should ensure that systems for leak and waste detection are included in the network design to assist the operators in locating and dealing with leakage and wastage. The Designer may also propose techniques for assessment of consumption; the current system, based on universal metering, has several drawbacks, such as meter tampering, defective meters, false/incorrect readings and meters that are not calibrated. All these increase the non-revenue water for the operators of the water systems.

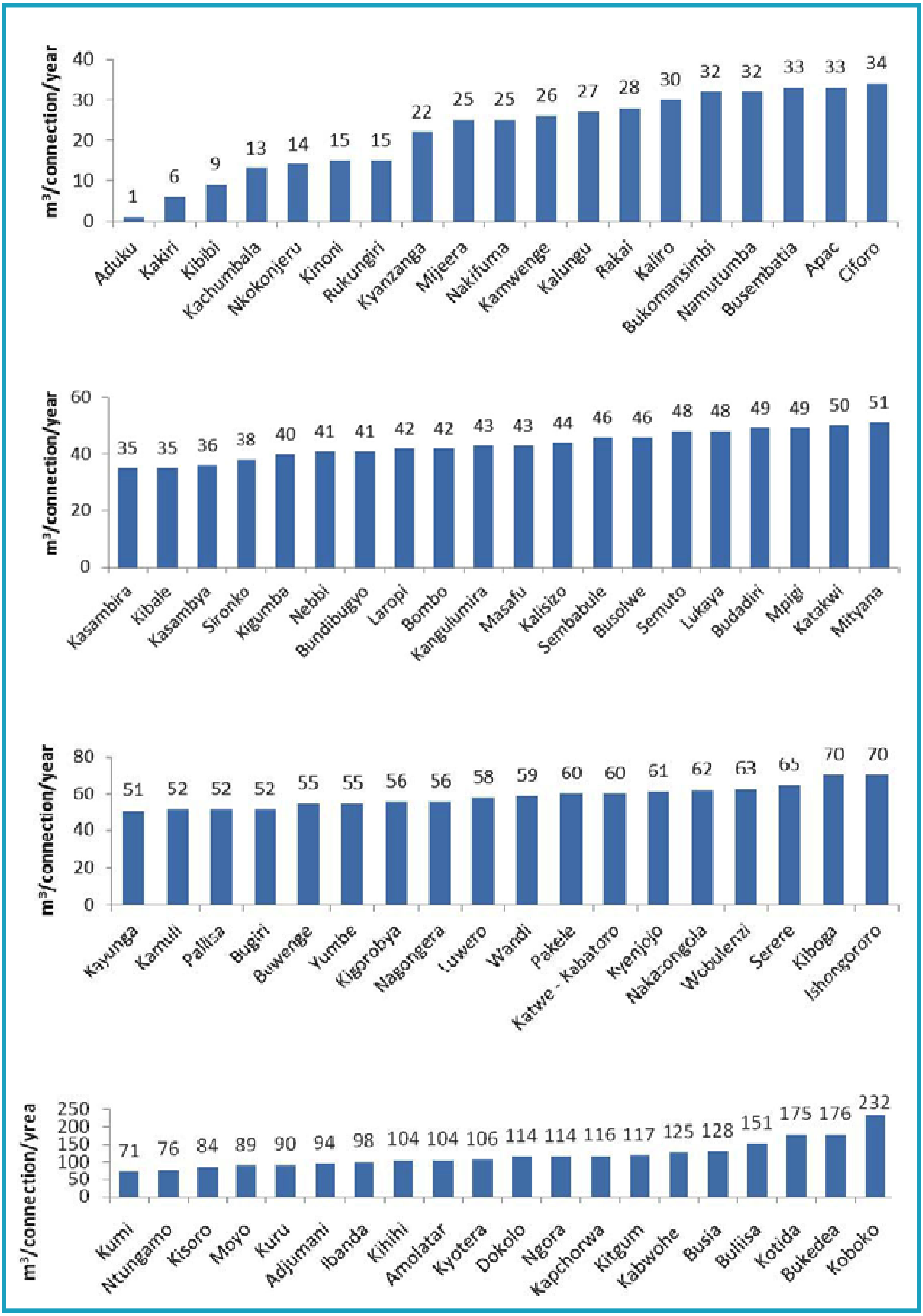


Figure 1-3 Unit Consumption for Small Towns under DWD (MWE,2011)

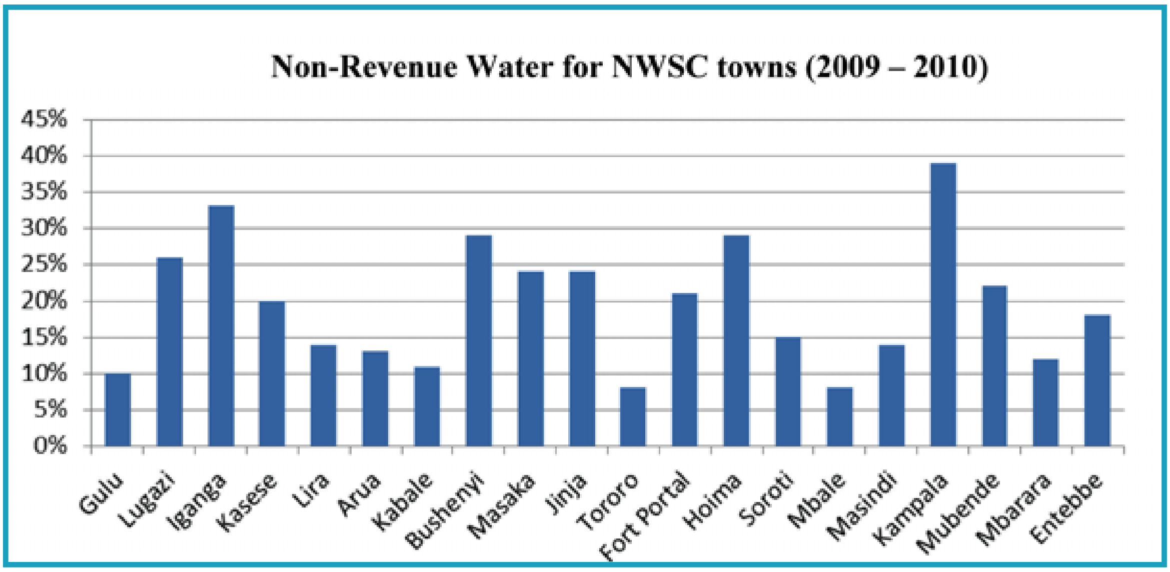


Figure 1-4 Non-Revenue Water for the NWSC Towns (NWSC, 2010)

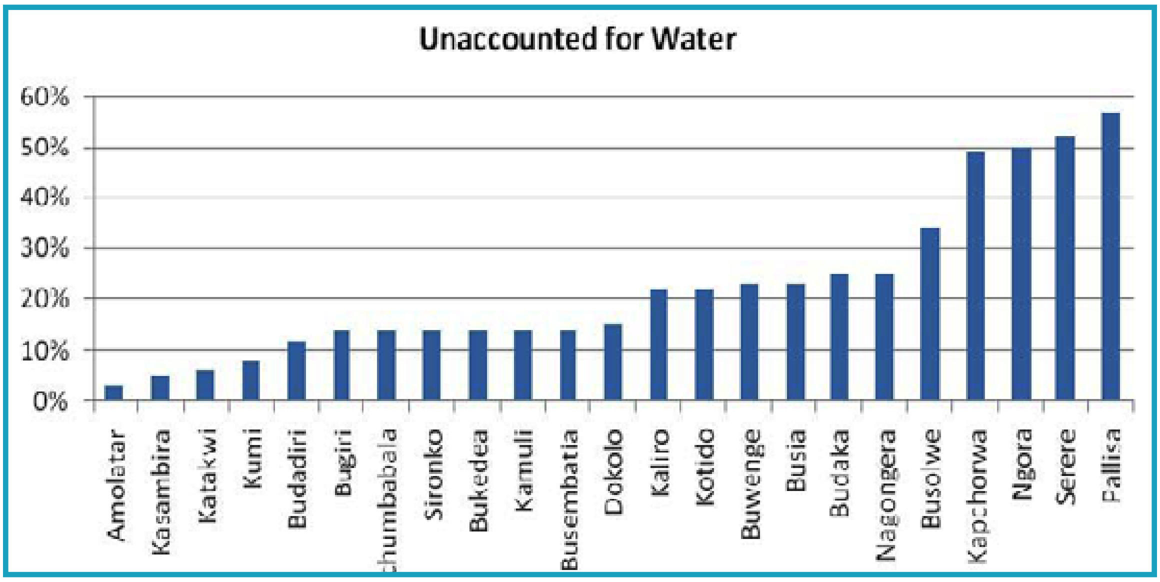


Figure 1-5 Unaccounted for water (non-revenue water) in Small Towns and RGCs
Source: DWD, 2011 (from a workshop presentation).

1.12 Coordination of Water System Management with Catchment Protection Efforts

Due to the increasing pollution of catchments that are used for water sources, it is becoming imperative for the water utilities to practice watershed protection to ensure wholesome raw water supplies. The costs of pollution control and treatment of polluted water are increasing rapidly since a catchment that is not protected brings in increasing pollution to the water source. Holistic catchment management should therefore be practiced right from the community level.

Once a micro-catchment is identified as a potential water source, all efforts should be made to protect and conserve the area for the future water source. For lakes and rivers, these critical areas, which should be protected, should be acquired and gazetted by the District or Municipal authorities as early as possible; resettling or acquiring developed (and therefore polluted catchments) is an option that should be avoided as much as possible. It is therefore important for the District and Municipal authorities to carry out surveys to identify these micro-catchments as part of the wider planning of their areas, in accordance with the Water Act, the Town and Country Planning Act and related legislation.

1.13 Unit Rates and Cost Estimates

The use of unit rates is well developed in the sector, with drilling contracts leading the way. Framework contracts for drilling of boreholes have been used for a number of years. The main advantage of the use of framework contracts is the assurance of price stability and the quick mobilisation of the contractors where there is work. Each contractor is assured of a reasonable volume of work. Repetitive tendering with its attendant delays is avoided meaning that the implementation of the sector programs can be done effectively.

The cost of inputs has fluctuated, sometimes with very large variations (mainly increases) such as with fuels and lubricants. Mechanisms should be built into the framework contracts to allow for price escalation (or reduction).

Per capita costs for new and old water supplies have been used by the Ministry to guide the budgeting process. The average per capita investment cost for 12 towns implemented during FY 2010/11 was US$ 40 [MWE, 2011]. These figures should assist the Designer in deciding options for development during the feasibility study stage. With all factors being equal, the unit cost should be adopted as the upper limit of investment in the area; however, local geographic, environmental and strategic considerations may modify the unit per capita investment cost. This justification should be included in the feasibility study report.

1.14 Regional Variations in Water Supplies in Uganda

Uganda is a country that has large variations in water resources potential, with some areas, like the Cattle Corridor, being water stressed, while others, such as those bordering Lake Victoria having heavy rains most of the year round. The Uganda Water Atlas, published by MWE, is a good reference for these variations [MWE, 2010 (2)].

In the mountainous areas of Uganda such as the Mt. Rwenzori and Mt. Elgon areas, the potential for gravity scheme development is abundant. Drilling of deep boreholes in those areas is impractical due to the difficulty in accessing the steep slopes with the heavy drilling equipment.

In the plains of South Western, Eastern and Northern Uganda, the potential for groundwater development is quite good and these would be the preferred water sources. However, the regional potential needs to be assessed adequately, especially taking into account the long term potential of ‘production’ boreholes in relation to climate change and the shortage of reliable aquifer capacity data.

1.15 Climate Change and Climate Variability

1.15.1 Introduction

Change in climate may potentially alter the frequency, quantity, location and duration of hydrological regimes. Changed hydrological extremes will have significant implications on the planning and design of hydraulic structures and general characteristics of water resources such as quantities and qualities (Grum et al., 2006, Nyeko-Ogiramoi, 2011). The changes could aggravate periodic and cyclic shortfalls of water supply in addition to other factors such as landuse malpractices.

1.15.2 Change in Intensity flow duration Curve

In the case of river hydrology, the design of hydraulic systems whose design period is comparable to the time scale associated to the induced climate change has traditionally been based on statistical analysis of extreme events extracted from historical records.Under the influence of climate change, stationerity assumption of must be reconsidered to account for the expected changes in the extreme precipitation events such droughts and floods (Grum et al., 2006). Thus, it is important to isolate the extremes from the overall rainfall distribution and assess how the projected climate change will affect the extremes for different aggregation levels or time spans over which the mean rainfall intensities are obtained. With the possible amplification of extremes by anthropogenic-induced climate change and the likely worrying projected situation, application of extreme value analysis to analyze rainfall extremes should be able to provide information that put the likely impacts into context. IDF is also applied in the design of water storage infrastructures.

1.15.3 Change in flow-duration-frequency

In river engineering applications, such as the designs of culverts and other related hydraulic structures, or water abstractions from a river or lake for various consumption, the relationships between flow, duration and frequency (Flow-Duration-Frequency or QDF) for both low flow (Water Supply design)and high flow (bridges, culverts, irrigation structures, etc.) are paramount. The third element of QDF, the return period (design period), provides crucial information on the frequency of occurrence of a given flow. Such information is very important for the assessment of climate change impacts on water availability hence on water engineering design. Like IDF, QDF can be analyzed based on Extreme Value (EV) theory, which forms the theoretical stochastic framework for the estimation of extreme quantiles. The parameter-QDF relationships, together with the analytical description of the extreme value distribution, for selected return periods, comprise the QDF curves. Information on the possible change in the QDF curves, because of the possible change in climate is significant for water resources engineering applications. To obtain such information, the flow peaks (low and high)can be fitted into extreme value probability distribution and the QDF curves for the control (from observed data) and that of the projected data (climate model data) runs can be compared to provide insights on the possible shifts in the observed QDF curves.

1.15.3 Flow duration curve

The Flow Duration Curve (FDC) is a plot of flow frequency versus return period of non-exceedance probability. FDC is normally used in determining the flow which is not exceeding once in a specified period of time or return period. It is also applied while designing storage capacity of reservoirs. The amount of water to be extracted from a river/spring (design flow) for supply for different purposes is normally determined based on FDC. Under climate change the design flow needs to be scaled by a certain climate change factor depending whether future climate (rainfall, flow extremes) is projected to increase or reduce.

1.15.4 Change in recharge

Change in groundwater recharge can be influenced by change in climate. This may affect the yield of an acquifer which is normally used to determine pump installation depth in deep and shallow water wells. Certain climate change factors need to be applied to drilling and pump installation depths to account for future climate change. These factors are normally the ratio between projected climate (future recharge value) and presence (reference) climate (current recharge value). This information can only be obtained by carrying out climate change impact assessment on recharge.

1.15.5 Application of change factors

When carrying out engineering designs it may proof difficult for the implementer of the project to carry out detailed impact assessment because it requires expertise knowledge and rigorous climate change impact modeling. An alternative way is to look for change factors from research works already conducted by scientist that cover the catchments or the study area. For example, the research study by NyekoOgiramoi (2011) on the impacts of climate change on water resources in Upper Nile basin (lake Victoria basin) indicated the following:

- Rainfall extremes are projected to increase by about 13-31%. The rainfall extremes for the western and southern parts of the L. Victoria basin are projected also to increase by 10-19%. In contrast, the rainfall extremes for the lower River Kagera catchment are projected to decrease by about 25% in the 2050s and 2090s.

- Moderate to high extreme events, including flood events (annual maximum streamflow) are projected to change differently. Streamflowofrboth the River Katonga and Ruizi, the medium extreme flow events are projected to be similar to the current situation in the 2050s but will decrease in the 2090s. The intensity and frequency of the annual maximum flow(AMF) series are projected to change differently as well. For the River Katonga, a decrease in the mean of the AMF by about 20% is projected for the 2050s but in the 2090s, a gain to the current state is projected. For the River Ruizi, a decrease is projected by about 5% in the 2050s and 15% in the 2090s. The possible change in the seasonal mean flow is projected to decrease by 20% and 15% for the Katonga and Ruizi rivers, respectively, which is proportional to the change in their respective MAF.

Figure 1-6shows a decision tree for incorporating projected climate change information into longer-term project planning. The chart shows potential scoping questions and answers leading to recommended options for considering whether and how to incorporate projected climate change information into project-specific planning.

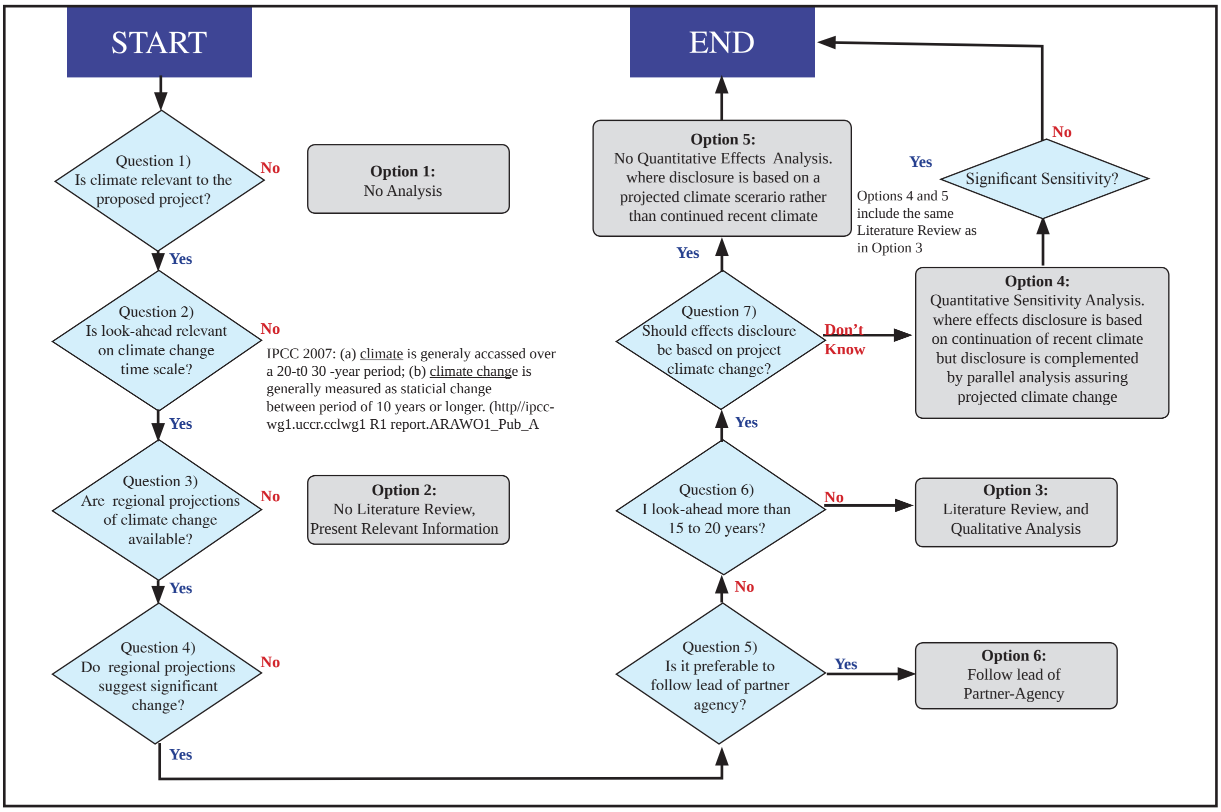


Figure 1-6 Decision Tree to Incorporate Climate Change in Design
Adapted from U.S. Department of the Interior (DOI) 2008

1.16 Right of Way for Water and Power Supplies

It is a recognised fact that water and power supplies have right of way; it is also a fact that landowners can and often do, object to passage of pipes and construction of appurtenances in their land. A designer of a water supply system should therefore be guided by the provisions of the Water Act and the Electricity Act.

Section 14 (i) of the Water Act gives the Director of DWD or a person authorised by him to 'enter and remain on land for purposes of performing functions or exercising powers conferred under the Act and may take such measures and construct or operate works as may be necessary for the investigation, use, control, protection, management or administration of water'.

Section 14 of the Water Act gives the obligations of the person exercising the powers of entry into land (a) cooperate as much as possible with the owner and occupier of the land; (b) cause as little harm and inconvenience as possible; (c) stay on the land only for as long as is reasonably necessary; (d) remove from the land, on completing any works, all plant machinery, equipment, goods or buildings brought on to the land, other than anything that the owner or occupier of the land agrees may be left there; (e) leave the land as nearly as possible in the condition in which it was prior to entry being made.

Section 18 (2) of the Water Act requires that person wishing to construct any works or to take and use water may apply to the director in the prescribed form for a permit to do so.

Section 67 (1) of the Electricity Act gives a person licensed by the Electricity Regulatory Authority either generally or on a particular occasion gives the authority to place and maintain electric supply lines in, over or upon any land and for that purpose it shall be lawful, upon written authorisation by the authority, for the licensee or his or her representative—(a) at all times, on reasonable notice, to enter upon any land and put up any posts which may be required for the support of any electric supply lines; (b) to fasten to any tree growing on that land a bracket or other support for the line; (c) to cut down any tree or branch which is likely to injure, impede or interfere with any electric supply line; and (d) to perform any activity necessary for the purpose of establishing, constructing, repairing, improving, examining, altering or removing an electric supply line, or for performing any other activity under the Act.

Section 67 (3) of the Electricity Act requires the licensee to do as little damage as possible to the land and to the environment and shall ensure prompt payment of fair and adequate compensation to all interested persons for any damage or loss sustained by reason of the exercise of the powers under this section. However, Section 67 (4) requires the licensee to give notice except for purposes of maintenance of the electric supply line and the owner might object to the entry in accordance with Section 67 (5).

1.17 The Water Supply Manual in Relation to the Project Cycle

1.17.1 Introduction

The Water Supply Design Manual covers only the design of water supplies, not the entire project cycle. A typical project cycle for water supplies is described in the typical Development Partner-supported program in Figure 1-6**.** The key water supply design project stages are prefeasibility or identification stage; feasibility and preliminary design stage and final design stages, which are elucidated below. Depending on the Funding Agency, there will be communications and approvals required at various stages of the cycle.

1.17.2 Prefeasibility or Identification Stage

The prefeasibility stage of the project cycle covers the identification stage whereby the need for a particular project type is broadly discussed at desk level and options decided on. Data from large scale maps, GIS maps, historical river flows and climatic information will form the basis for a prefeasibility assessment. Broad budgets, population estimates and implementation schedules may be drawn and funds may be sourced for the future projects.

Identification of projects is usually done in-house by the Implementing Agency as part of the sector-wide and multi-sector development planning processes; however, identification should be informed through studies such as water and sanitation coverage, status of towns (some may become district headquarters) and populations. Multiple use projects (for example agriculture and water supply) may be developed, including the phasing of the overall program and is called the sector strategic investment plan (SSIP).

Environmental scoping or strategic environment assessment should be carried out to inform the development planning processes.

The timeframe for prefeasibility of water and sanitation projects is typically 5 – 30 years before some of the identified projects advance to the feasibility and preliminary design stage.

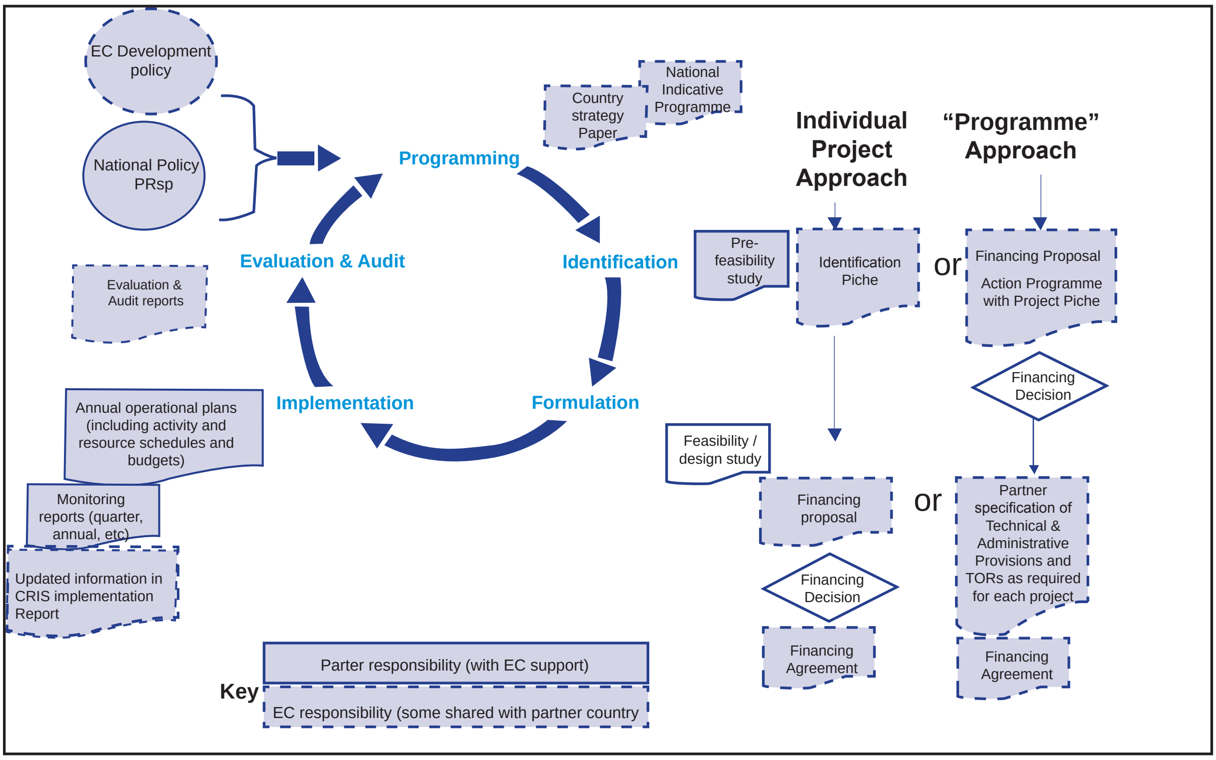


Figure 1-7 EU Project Cycle [EU, 2004]

1.17.3 Feasibility and Preliminary Design Stage

The feasibility preliminary design stage follows from the prefeasibility stage and involves field measurements, preparation of accurate estimates of flows and yields of water sources and their water quality, population estimates, implementation modalities and schedules; preparation of accurate budgets, financial and economic assessments and environmental and social impact assessments. The environmental and social impact assessments should be submitted to the responsible authorities for approval and resettlement action planning should commence at this stage. The feasibility stage must be narrowed down in scope and budget so that the Consultant arrives at reasonable and implementable options.

The Design Team typically plans for the feasibility and preliminary design stages as follows:

i) Consultation with the Implementation Agency (Client) in order to get first-hand information and data for the Poject Area. Any expected obstacles to the design process may be disclosed at this time, such as difficult terrain, expected resettlement actions and environmental challenges, the political climate in the area and the time schedules expected. Data requirements include flow and water quality records of streams and rivers, hydrogeological data for any boreholes in the area (obtainable from DWD and DWRM); population and socioeconomic data (available from UBOS); rainfall, sunlight and other weather information (available from the Meteorological Department); survey data (available from the Department of Surveys) and contacts of the key persons in the Project Area and district offices. The Consultant should also obtain any design reports and other relevant studies that have been made for the Project Area water supply both by the Client and other stakeholders, such as NGOs;

ii) The Design Team will make an inception field visit to the Project Area. The Design Engineer, Socio-economist, Environmentalist, Hydrologist, Hydro-geologist, etc. will be required to visit the sites and make assessment of the potential sources (in terms of location, yield, water quality, access, environment etc.) and the potential service area (core and fringe) and the potential population. Core areas of a town are the central business district and other built up areas within the town boundaries while the fringe areas are built up areas outside the town boundaries. The Socio-economist will estimate the population within the core and fringe areas using sampling techniques and data available from the Project Area and UBOS. Categories of demand by the population (domestic, institutional, industry, etc.) will also be determined. A survey of the area might be necessary but handheld global positioning systems (GPS) can suffice for most measurements;

iii) Siting of water sources (for groundwater and for surface water) should be done after the inception visit. Depending on the demand estimated over the project lifetime, demand may be met by groundwater or surface water, or both (especially if implemented in a phased manner). Measurement of the flows of surface water sources should be conducted over a long time, though this is not always possible. The Hydrologist should make use of available stream flow data and interview people who live near the source for historical trends, with the aim of determining if the surface water source dries up during the dry season, or has significantly lower flow that would not be sufficient for the Project Area supply. Measurement of water quality should be done to ascertain whether the minimum water quality standards for raw water are met;

iv) Where potential borehole sites are determined, the Hydro-geologist should supervise drilling of production boreholes and determine the sustainable borehole yields and water quality. Contracting of the drilling contractor requires the Client to tender out the works and since this may take a long time unless the Client has running contracts with drilling companies;

v) The Design Team should make outline designs for several options: make drawings, bills of quantities and carry out financial and economic analyses. Options that are obviously unfeasible, such as where the stream dries up during the dry season, or the potential source of surface water is extremely distant or where the water is known to be salty, should be dropped or ranked low; and

vi) The Design Team will compile the information and analyse the suitability of the sources for supplying the population projected with the appropriate service levels. With all the analysis complete, the Design Team must rank the feasible options according to clear criteria, such as yield of the sources, environmental and social impacts, financial and economic cost, areas to be served, unit cost of the overall project, etc. and make recommendations of the best three options to the Client.

The time schedule for feasibility and preliminary design stage is typically 2 – 5 years before the project moves to the next stage.Large water supply projects take longer to reach the next stage since they are often funded by Development Partners whose approvals may take time.

The exact time schedule therefore is dependant on the scale and complexity of the project.

1.17.4 Detailed Design Stage

The detailed design stage follows the feasibility and preliminary design stage. The environmental and social impact assessments should have been approved by this stage so that the option which is implemented during the next stage has been fully approved by the responsible authorities. Implementation of the resettlement action plan should be done early during this stage so that the project affected people do not interfere with the design and construction stages. The stage will include preparation of detailed designs, detailed drawings, bills of quantities, tender documents, estimates of the construction cost (Engineer's Estimate) and the detailed implementation program (work plan) to guide the project implementers.

The Design Team typically plans the design (guided by the terms of reference from the Client) as follows:

i) Consultation with the Implementation Agency (Client) in order to get first-hand information and data for the Project Area. Any expected obstacles to the design process may be disclosed at this time, such as difficult terrain, unresolved resettlement actions, the political climate in the area and the time schedules expected;
ii) Acquisition of data such as water quality, geotechnical data, land survey data and socioeconomic and environmental data from the feasibility and preliminary design stage. This assists the Team in determining the inputs from each of the specialists and also to make fieldwork and consultation plans;
iii) Inception field visit should be conducted to enrich the inception report. During the inception field visit, the Design Team will make rapid assessments of the facts laid out in the feasibility study report and update where necessary. The boundaries of the service area are determined and the survey team may commence with locating the roads, water sources, tanks and other appurtenances and the water supply points (stand pipes, house connections, institutions, industries etc.) working closely with the socioeconomic and engineering teams. Siting of water sources, if not done already, should also commence;
iv) The Design Team will need a second visit to the Project Area in order to improve on the details, such as siting of water sources and drilling supervision, or acquisition of land for surface water works. The population data will be completed and the service levels finalised so that the final demand projections are made. Once the final water sources are known, the Surveyor incorporates them into the general Project Area maps that should have been prepared, allowing the Design Team to consult with the local authorities and the Client over the final coverage of the project. Due to limitations such as topography, distance and funding, some of the communities might not be covered to the desired levels. The Client must be kept abreast of these developments;
v) It is recommended for the Design Team to set up an office in the Project Area with a drawing office from where layouts, profiles and other essential details can be printed and discussed. This saves on repeated travel to the Project Area to rectify issues that may arise as the designs are developed. At the end of the second field trip, a first draft design should be prepared and included in the draft report to the Client; and
vi) The final design should have all the components of the terms of reference, such as drawings, bills of quantities, cost estimates and tender documents.

The time schedule for the detailed design stage is typically 1 – 2 years before the project progresses to the next stage.

1.17.5 Construction Stage

Approval of the detailed designs by the Implementation Agency (Client) and availability of funding from the Funding Agency leads to the tendering of the works for construction.

1.17.6 Operation and Maintenance Stage

Following the completion of the project and rectification of defects, the project enters the operation and maintenance stage where the owner or operator runs the project. For water supply projects, the systems is often tendered out to a private operator who will operate and manage all aspects of the system, including treatment of water, pumping to mains tanks, distribution to consumers, billing and collection of revenue, control of leakage and waste and implementing other aspects as required in the contract.

A key aspect is reporting on the parameters that are required in the owner's performance contract. The performance contract is a delegated responsibility to lower local governments as required in the Water Act.

1.17.7 Monitoring and Evaluation

During Operation and Maintenance, there should be means to check progress (**monitoring**) and take stock of where things are at on a regular basis (**evaluation)**.Monitoring is performed while a project is being implemented with the aim of improving the project design and functioning while in action, well as evaluation studies the outcome of a project with the aim of informing the design of future projects (Bamberger et al., 1986).

Evaluation and monitoring systems can be an effective way to:

i. Provide constant feedback on extent to which the projects are achieving their goals;

ii. Identify potential problems at an early stage and propose possible solutions;

iii. Monitor the accessibility of the project to all sectors of the target population;

iv. Monitor the efficiency with which the different components of the project are being implemented and suggest improvements;

v. Evaluate the extent to which the project is able to achieve its general objectives;

vi. Provide guidelines for the planning of future projects;

vii. Influence sector assistance strategy. Relevant analysis from project and policy evaluation can highlight the outcomes of previous interventions, and the strengths and weaknesses of their implementation;

viii.Improve project design. Use of project design tools such as the logframe (logical framework) results in systematic selection of indicators for monitoring project performance. The process of selecting indicators for monitoring is a test of the soundness of project objectives and can lead to improvements in project design;

ix. Incorporate views of stakeholders. Awareness is growing that participation by project beneficiaries in design and implementation brings greater "ownership" of project objectives and encourages the sustainability of project benefits. Ownership brings accountability. Objectives should be set and indicators selected in consultation with stakeholders, so that objectives and targets are jointly "owned". The emergence of recorded benefits early on helps reinforce ownership, and early warning of emerging problems allows action to be taken before costs rise; and

x. Show need for mid-course corrections. A reliable flow of information during implementation enables managers to keep track of progress and adjust operations to take account of experience (OED).

1.18 Updating of the Water Supply Manual

The Manual is a living document and should be updated, improved and expanded on a regular basis. A 10-year period between revisions should be sufficient (the first edition was prepared in

2000 and between it and this revision is 12 years). The Manual is designed to guide the water supply design process and not to replace the standard water supply design textbooks. Factual errors should be brought to the attention of the Editorial Committee by the user, by contacting them through the Design Review Committee of the Ministry of Water and Environment.

1.19 Glossary of Terms Used in Water Supply

Word or Phrase	Meaning Usually Attributed in Water Supply
1. Ability to pay	An economic principle stating that the amount of water bill an individual pays should be dependent on the level of burden the water bill will create relative to the wealth of the individual. The ability to pay principle suggests that the real amount of water bill paid is not the only factor that has to be considered, and that other issues such as ability to pay should also factor into water tariff system.
2. Community Letter of Agreement	An agreement made between the WA and the community for the grant of land for the development of water supply infrastructure such as water source, water tanks and for Public Standpipes.
3. Demand	The amount of water used by a consumer in a day, expressed in m^3/d.
4. Design Period	The length, in years, over which the water supply system is meant to supply adequate water.
5. Effective Water Depth	The depth inside a sedimentation tank net of the sludge zone.
6. Future Year	The year in which the newly constructed water supply system will reach 10 years after the Initial Year.
7. House Connection	A point of supply meant for use by a household and is not used for sale of water.
8. Initial Year	The year in which the newly constructed water supply system will start operating, assumed to be 5 years from the date of commencement of the feasibility studies.
9. Large Town	A major town with population of over 15,000 people.
10. Livestock Equivalent	The amount of water consumed by a livestock equivalent expressed in litres/day. It is equal to 50 litres per day.
11. Low Flush Toilet	A water cistern that uses low amounts of water for flushing compared to the common flush toilets.
12. Micro-catchment	The immediate catchment, usually part of a larger catchment that is the minimum area to be protected for a water source.
13. Non-pipe Supply	Areas where supply is by trucks or other means.
14. Person Equivalent (PE)	The amount of water consumed by an equivalent of one person. It is equal to 50 litres per day.
15. Public Standpipe	A point of supply for a community that serves more than a household and is often for sale of water.
16. Rural Growth Centre	A rural town with population from 500 – 5,000 people.

Word or Phrase	Meaning Usually Attributed in Water Supply
17. Rural Industries	Industries located outside the gazetted principal urban centres.
18. Service Level	This is a category or standard of water supply service. The service level includes public stand pipe; yard tap; house connection and non-piped supply.
19. Small Town	A rural town with population from 5,000 to 15,000 people.
20. Ultimate Year	The year in which the newly constructed water supply system will reach its design life, assumed as 20 years after the Initial Year.
21. Uniformity coefficient (of filter media)	The ratio of the size at which 60 per cent (by weight) of a sand sample passes through a sieve (in other words 60 per cent of the sand is finer than a given size) divided by the size at which 10 per cent of the same sample (by weight) passes through a sieve (10 per cent is finer than a given size). A UC of 1 indicates all the particles are the same size. As the number goes up the size differentiation becomes greater and the quality of the sand becomes less desirable for use in a slow or rapid sand filter.
22. Walking Distance	The distance that consumers travel to the nearest water supply point for the appropriate service level.
23. Water Authority	A body gazetted by the Minister in charge of water to be responsible for the water and sewerage services areas in accordance with the Water Act.
24. Willingness to Pay	A measure of demand for particular levels of water supply and/or sanitation service. It is often assessed as part of a contingent-valuation study, in which demand for service improvements at the community level is estimated.
25. Yard Tap	A point of supply meant for the use of a household and neighbours and is often used for sale of water.

Chapter 2

WATER DEMAND

2.1 General

Estimation of the water demand for a community is the first activity of the design process. It starts with the delineation of the proposed supply areas and followed by a socioeconomic baseline survey. This survey involves counting the number of potential users. It is often impractical to count all the potential consumers; however, statistical methods and sampling, information obtained from UBOS, from the district and local councils may be used to improve on the estimates. Because water supplies are meant to serve for several years, the future population is also required.

A water supply scheme will almost without exception, cater for water demand which will be increasing continuously with the years to come. When designing a scheme, a decision has to be made regarding the time in the future for which the various components of the scheme are to be designed.

To calculate the water demand for a water supply scheme, it is necessary to:

i) Determine the numbers of consumers falling within the different consumer categories at various stages of the design period.
ii) Determine the average day unit water demand figures for the various consumer categories concerned.

The common categories of consumption are domestic (which are further categorised into high, medium and low income categories); commercial and institutional categories. In each category, consumption should be estimated and projected independently.

2.2 Design Period

The design period is the period within which the project long term projected demands are estimated for a least cost project. The design period of the project is the function among other things of the discount rate (opportunity cost) and the economy of scale factor. It is generally accepted that the optimum design period is between 5-10 years and should rarely exceed 20 years. Normally, water demand projections should be made for the "Initial Year" the "Future Year" and the "Ultimate Year".

Table 2-1 Design Period

Design Period	Duration from Commencement	Narrative
Initial Year	5	This is the year when the water supply scheme is expected to be commissioned into operation, which may be assumed to be 5 years from the date of commencement of the feasibility studies.
Future Year	15	10 years ahead of "Initial Year".
Ultimate Year	25	20 years ahead of "Initial Year".It is normally based on life expectancy of electro-mechanical components.

Once the "Initial Year", "Future Year" and "Ultimate Year" have been determined for a scheme, they should not be changed.

Future population figures are then projected and by judgment of the future growth possibilities of the supply area, the best figure is selected. The choice of method to be adopted in each particular case will depend upon the nature of the supply area, habits of the people, in or out migration, scope for future expansion etc.

The formula to be used is as follows:

$$P_n = P(1+r)^n \qquad \textbf{Equation 2-1}$$

Where

P = population after n years,
P^n= present population, and
r = annual growth rate (%)

A water supply scheme should normally be designed for the "Ultimate Year" demand. However, phasing of the implementation will often prove necessary from a financial viewpoint, and the possibilities of phasing should therefore be examined against the background of the "Initial Year" and the "Future Year" water demand projections.

Electrical and mechanical equipment is normally designed for shorter periods than civil engineering and building works, a reflection of the different "service life"/"economic life" periods for the different water supply scheme components.

2.3 Domestic Demand

2.3.1 Population Projections

The present populations in a proposed water supply scheme area should be estimated based on the latest census estimates. However, the results should be cross-checked with information obtained from other sources, such as the Local Council (LC) authorities. The population in each Local Council I (LC I) area should be projected separately. In the instance where UBOS and LC population figures give contradictory results, the survey must count the number of potential users. Population figures for towns should be confirmed by physically counting the houses, shops, bars, etc. in the towns. Where the numbers are large, a representative sample should be chosen.

Populations in large towns and urban centers should be analyzed separately for different areas and different income categories. "High", "medium" and "low" income housing populations should be projected independently. The projection of future populations can be quite difficult. Therefore, all available demographic information should be collected and evaluated.

The following sources of information, wherever applicable, will be useful in determining the likely future population growth rates.

i) The growth in population which has taken place in the area in the past. Compare figures of the 1969, 1980, 1991 and 2002 censuses and the respective inter-census population growth rates as shown in appendix 1. It should be noted that District and sub-county names and boundaries might have been altered during the inter-census periods;
ii) District Development Plans (DDPs);
iii) Town council physical development plans;
iv) Official projections by the Statistics Department of the Ministry of Finance and Economic Planning; and
v) Opinions of the Local Administrations.

Uganda Population Pyramid for 2010 and projection for 2020 and 2050

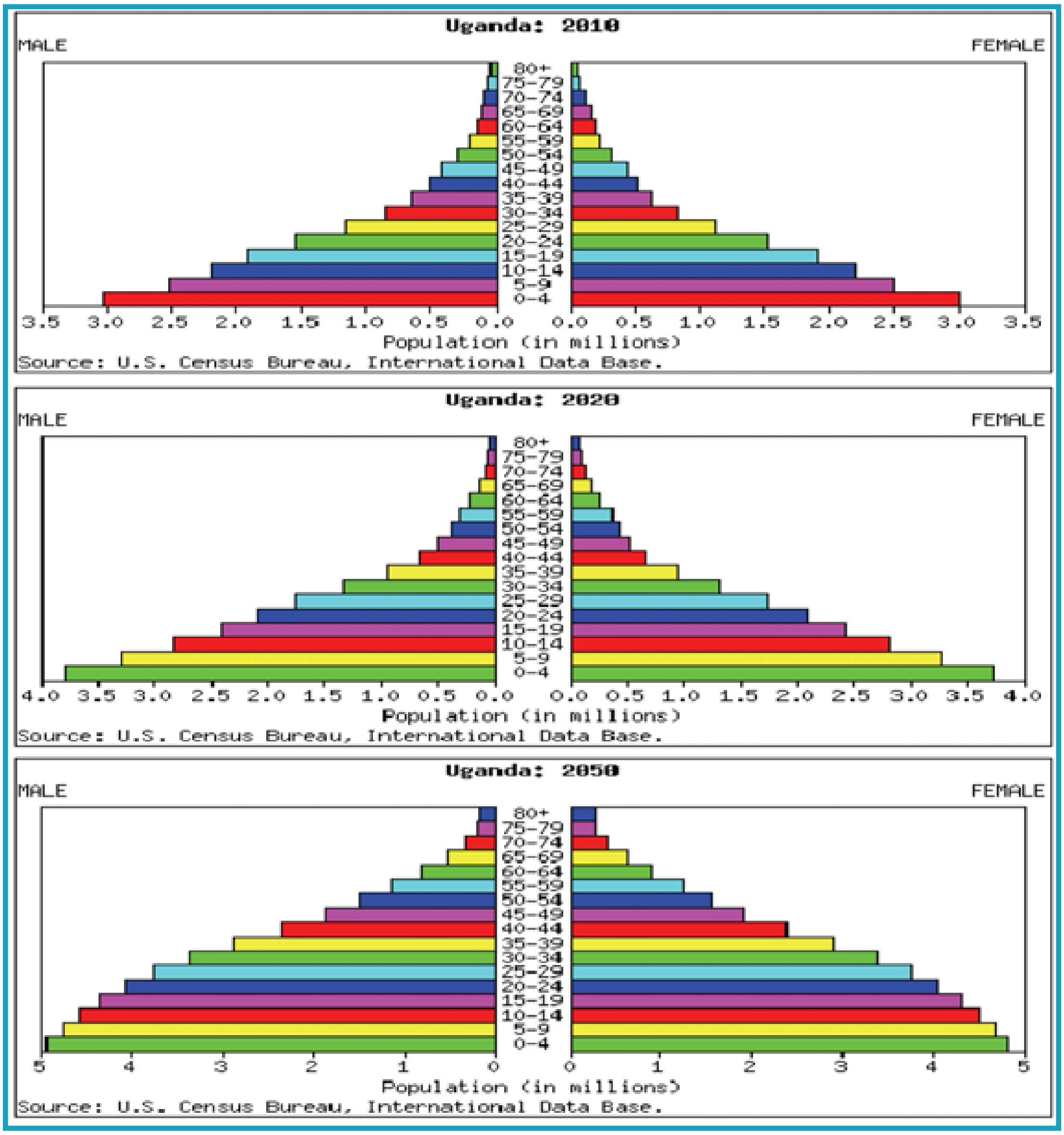


Figure 2-1 Age and sex distribution for Uganda

There has been a substantial subdivision of the districts as such the latest population figures should be obtained from the most current census figures available from UBOS or other credible sources should be used in the design. The population growth rates should be updated with current information available from UBOS and from locally available information. The list of sub-county populations is given in Appendix 1.

2.3.2 Unit Costs and Service Levels

The unit costs are derived using the Unit Cost Estimating Tool that is built up by examining typical bill of quantities and is based on the common variable of m^3/day capacity of the system components. The costs in a particular district (for small towns) is determined by the technology mix in that district which for urban services will predominately be determined by the size of the towns as the per m^3 cost of supplying water depends chiefly on the size of the scheme. For the NWSC investments the unit costs are determined for the individual systems.

The per capita unit costs depends on the service level mix – the proportion of persons served through house connections, yard taps and public standpipes respectively as the consumption of water depends on the type of connection. Target values for the per capita consumption will be set and a composite per capita cost will then be calculate depending on the size of towns and type of technology.

The per capita consumption used in the small towns are 20 l/day for small towns up to 5,000 persons, 35 l/day for medium towns up to 20,000 persons and 50 l/day for the larger towns. The per capita consumption estimates include the institutional and commercial use and therefore increasing with the towns size since there are more institutions and commercial activities in the larger towns. The NWSC estimates are based on data from the individual systems on number of connection, production volumes etc. the water demand per consumer category is as given in the tables below.

Table 2-2 Demand per Consumer Category

CONSUMER CATEGORY	Rural Area (l/ca/d)	Urban Area (l/ca/d)	Remarks
Low income using kiosks or public taps	20	20	Most squatter areas, to be taken as the minimum
Low income multiple household with Yard Tap	40	40	Low income housing with no inside installation.
Low income, single household with Yard Tap	50	50	Low income housing with no inside installation.
Medium Income Household		100	Medium income group housing, with sewer or septic tank.
High Income Household		200	High income group housing, with sewer or septic tank.

The determination of the three house connection categories of high, medium and low income categories is loosely linked to the number of water using gadgets in the houses. In practice, it is not possible to distinguish exactly how many of each house type are in a village. Estimatesbased on sampling and historical data should be used. The demand per consumer category is presented in Table 2-1 and the results of a socioeconomicsurvey of Kampala area is given in Table 2-2.

Table 2-3: Kampala NWSC Customer Income Ranges.

Income Category	Income Range (2004) UGX	Income Range (2008) UGX
Low	Less than 43,000	Less than 503,000
Medium	43,000 – 1,200,000	503,000 – 1,403,000
High	More than 1,200,000	More than 1,403,000

Source: Kayaga and Franceys, 2007.

Service levels should be derived from the income levels, the “Ability-to-pay” and the “Willingness-to-pay” of the local communities, as determined from the socioeconomic study. “Ability-to-pay” and “Willingness-to-pay” will in turn depend on the proposed tariff structures and levels.

When designing expansions to existing schemes, the distribution between House Connections, Yard Taps and Public Standpipes should be determined based on observations of the actual situation on the ground. The up-take of private connections is an important factor in indicating and stimulating the level of development in an area.

The maximum walking distance from a public standpipe to the dwellings served should normally not exceed 500 m in rural areas and 250 m in urban centres. The number of users of a public standpipe should normally be restricted to 300 persons in rural areas, and 250 persons in urban centres. In heavily populated areas, the walking distance will be reduced significantly to allow for the higher populations; conversely in sparsely populated rural areas, the walking distance may increase to 1 km. The World Health Organisation (WHO) routinely gives internationally accepted guidelines on service levels and should be consulted where possible.

The recommended figures for walking distances and numbers of users for public standpipes can be changed by the designer in exceptional cases to suit the actual situation prevailing in the supply area. However, this should first be discussed and agreed with the beneficiaries of the water supply scheme.

2.3.3 Educational Institutions

Water demand in educational institutions should be based on the situation prevailing at the time of scheme design, the development plans of the Ministry of Education and Sports, Ministry of Health, other large institutional consumers such as the Uganda Police, Uganda Prisons, the UPDF and the District Local Governments, and the projected growth of the local population.

For preliminary estimates, it may be assumed that 40% of the population attends primary and/ or secondary school. This is based on the population projections as shown in figure 2-1 and the fact that approx. 6-19 year range of Uganda’s population are school going age. Day schools and boarding schools should be analysed separately. Sanitary standards with regard to the use of water closets should also be studied. In this regard, the use of “Low Flush” toilets should particularly be encouraged.

The water demand for staff should be included in the total demand for the educational institution concerned. For isolated educational institutions, the neighboring areas might also access the water supply of the institution and should be taken into account.

Projection of institutional population growth should be done carefully. Schools and hospitals might not grow as rapidly as the surrounding population; however, new institutions or significant expansions (and rarely, closures) of institutions might occur over the design period. If a school has remained with the same population for a long time, it might not be prudent to estimate a high growth rate.

2.3.4 Health Institutions

Water demand in health institutions should be based on the situation prevailing at the time of scheme design, the development plans of the Ministry of Health and the District Local Governments, and the projected growth of the local population. In the National Development Plan (2010/11 to 2014/15), it is planned for about 20,000 people to be served by one health centre (HC) III (incorporating an out-patient’s department, a maternity and a few beds for in-patients), to be provided one in each sub-county.

In Uganda, the number of hospital beds can be assumed to be 1.2 per 1,000 people. However, district and other major hospitals should be studied in more detail separately. The water demand for staff should be included in the total demand for the health institution concerned.

Projection of the hospital population should be done on a case-by-case basis using realistic growth estimates and the development plans of the Ministry of Health and other sector players. It should not be assumed to grow at the same rate as the surrounding population since hospital facilities can grow rapidly during the expansion phase but might stagnate for long periods of time.

Table 2-4: Institutional Water Demand

CONSUMER	UNIT	RURAL l/d	URBAN l/d	REMARK
Schools Day Schools	l/std/d	10	10 20	With pit latrine With WC
Boarding Schools	l/std/d	50	100	With WC
Health care Dispensaries	l/visitor/d	10	50	Out patients only
Health Center 1	l/bed/d	20	50	No modern facilities
Health Center 2	l/bed/d	50	70	With maternity With pit latrine
Health Center 3	l/bed/d	70	100	With maternity With pit latrine
Health Center 4	l/bed/d	100	150	With maternity With WC
Hospital, District	l/bed/d		200	With surgery unit
Hospital, Regional Referal	l/bed/d		400	With surgery unit
Administrative Offices	l/worker/d	10	- 70	With pit latrine With WC

2.3.5 Other Institutions

Water demand for other institutions should be estimated based on the situation prevailing at the time of scheme design and the future development plans of the institutions. Wherever such data is not readily available, the assumptions used by the designer to estimate water demand should be stated clearly.

2.3.6 Micro Industry Demand

Commercial enterprises include shops, workshops, restaurants, bars, hotels, banks, etc. Water demand for commercial enterprises should be based on the situation prevailing at the time of scheme design, and the expected development based on the area development plans. In this regard, it can be assumed that future increases in commercial activity will be directly related to the growth in population. In many small towns and rural growth centres, the shops also double as dwellings and bars or pubs. The demand estimate should take this into account.

2.3.6 Industrial Demand

Water demand for industries should be studied in detail by consulting the proprietors concerned and other relevant agencies. Areas designated as “Industrial Areas” in the town council physical development plans but for which the exact nature of the industry is not known, should be allocated quantities of water per unit area as indicated in Table 2-5. However, realistic time frames for the gradual development of such areas must be considered.

Table 2-5: Micro Industry Water Demand

Industry	Product of Raw Material Unit	Water Consumption in m^3 per Unit of Raw Material
Food Industry		
Diary	Milk received (m^3)	2 - 5
Abattoir	Animals slaughtered (ton)	5 - 10
Brewery	Beer (m^3)	10 – 20
Sugar	Cane (ton)	10 - 20
Grain millers	Grain received (ton)	2 - 5
Others		
Pulp mill	Bleached pulp (ton)	100 – 800
Paper	High quality paper (ton)	300 – 500
Chipboard factory	Chipboard (ton)	50 – 200
Tannery	Raw skins (ton)	50 – 150
Cotton mill	Cotton thread (tufi)	50. – 150

Table 2-6: Macro industry Water demand

Industry Type	Water Demand m^3/ha/d
Medium Scale (water intensive)	40
Medium Scale (medium water intensive)	15
Small Scale (dry)	5

2.3.8 Water for Production Demand

2.3.8.1 Irrigation

Normally, water demand considerations should not include provisions for irrigation apart from very limited garden watering which, in any case, is already included in the per capita unit water demand figures. It is however not practical that treated water is used for large scale irrigation.

2.3.8.2 Livestock

According to the Water Act of Uganda 1997 in Part I – Interpretations, "livestock unit" means a mature animal with a live weight of 500 kilograms and for the purposes of this definition—

i) one head of cattle shall be deemed to be 0.7;
ii) one horse shall be deemed to be 0.6;
iii) one donkey shall be deemed to be 0.4;
iv) one goat shall be deemed to be 0.15; and
v) one sheep shall be deemed to be 0.15; of a livestock unit;

To cater for pigs and poultry farming, the figures below can be used for design:

i) one Pig shall be deemed to be 0.4; and
ii) one chicken shall be deemed to be 0.05 of a livestock unit;

Where demand is large, consideration must be made for bulk water transfers, covered under Chapter 7 - "Water Transmission and Distribution".

2.3.8.3 Aquaculture

Treated water would normally not be used for aquaculture. This topic will therefore be handled in the bulk water sections.

2.3.8.4 Rural Industries

Rural industries include paper mills; milk cooling plants; coffee hulleries; tea estates; soap manufacturers; fruit and juice processing plants; leather tanning plants; abattoirs; fish processing plants; soda and alcoholic beverage plants; flower farms; poultry and piggery farms, etc. Some of the rural industries may use treated water – such industries often modify the mains supply appreciably. Some industries, especially those that require water for cooling or cleaning, will require substantial amounts of untreated water. The demand for each industry should be assessed according to its specific requirements for both treated water and water for production.

2.3.9 Other Commercial Uses

Other demands include water sellers, such as water tenders; water for construction purposes (which may be high but transient – lasting for the length of the construction activities); water for street washing / cleaning, fountains and other municipal uses; water for recreation such as swimming pools or for water sports; etc.

Table 2-7: Other Commercial Water demand

Consumer	Unit	Rural	Urban	Remarks
Hotels/Lodges	l/bed/d	50	50 300 600	Low class Medium class High class
Bars/Restaurant	l/bar/d	500	700 1,000	Low class High class
Shops	l/shop/d	50	50 100	Small Town Large Town
Shops/dwelling place	l/shop/d	100	150 200	Small Town Large Town
Petrol Station/washing bays	l/Petrol Station/d	1,000	5,000 10,000	Small Town Large Town
Markets	l/ha/d	20,000		
Public Sanitation	l/person/d	20	50 70	Small Town Large Town

2.3.10 Fire Fighting

In the large towns, water demand for firefighting should be determined in collaboration with the relevant fire authorities. In addition to the quantity being available for firefighting, the location of fire hydrants should be carefully thought out and areas that are heavily built, industrial areas, large institutions and specific high rise buildings should have fire hydrants located in the vicinity. Emphasis should be made on accessibility of the fire hydrants to the fire trucks; however, they should also be secure from commercial water sellers who may misuse the hydrants by getting water free of charge.

For smaller urban centers, it is recommended that the capacity for fire-fighting should not be less than 10 l/s during a period of 2 hours.

Normally, there should not be any provision made for firefighting in rural water supply schemes. Where a supply is provided for a rural industry, consideration should be made for a fire fighting system, especially if the industry deals in inflammable items such as foam mattresses; paper; petroleum products; timber; etc. Provision may be made for pumping from an underground reservoir located strategically in the vicinity of the factory. Rural schools and other large institutions should have their own firefighting systems depending on the individual requirements. Water for firefighting is considered as part of Non-Revenue Water (NRW) and therefore forms part of the unbilled authorized consumption.

Design criteria for pipes and reserves should be as shown below

i) Minimum pipe diameters, flows, pressures and hydrant distribution
ii) Pipe sizes in the non-industrial firefighting distribution system should not be less than DN80 but where there are parallel distribution mains down both sides of a road, the distribution main without hydrants may be DN65. In mixed or general industrial areas the minimum diameter should be DN110/100 and on industrial estates, it should be DN150.
iii) The design flow into a fire hydrant should not be less than 10 l/s. The residual head in the pipes at a hydrant should not be less than 15m. The distance between two adjacent fire hydrants should not exceed 300m such that the distance of a building from a hydrant is not more than 150m.
iv) Firefighting reserve should be in accordance with the number of people served by a reservoir as shown in the following table below

However, since such a high rate is exerted for a short while by which time the fire either dies down or is brought under control and since the instance of fires are relatively few in number each year, the per capita water required is either calculated on yearly basis or may be ignored. The required number of fire hydrants should be provided at suitable locations in the distribution systems so that the specified number of fire hydrants can be used during the emergency according to the requirements.

Table 2-8: Fire Fighting Flows

<table>
<tr><th>Category</th><th colspan="2">Description</th><th>Flow in l/s</th></tr>
<tr><td rowspan="2">Housing</td><td colspan="2">Housing developments with units of detached or semidetached houses of not more than two floors should have a water supply capable of delivering a minimum through any single hydrant of:</td><td>8</td></tr>
<tr><td colspan="2">Multi occupied housing developments with units of more than two floors should have a water supply capable of delivering a minimum through any single hydrant on the development of:</td><td>20 - 35</td></tr>
<tr><td>Transportation</td><td>Lorry/coach parks, multi-storey car parks and service stations</td><td>All of these amenities should have a water supply capable of delivering a minimum through any single hydrant on the development or within a vehicular distance of 90metres from the complex of:</td><td>25</td></tr>
<tr><td rowspan="4">Industry</td><td rowspan="4">In order that an adequate supply of water is available for use by the Fire Authority in case of fire it is recommended that water supply infrastructure to industrial estate follows with the mains network on site being normally not less than DN150</td><td>Up to one hectare</td><td>20</td></tr>
<tr><td>One to two hectares</td><td>35</td></tr>
<tr><td>Two to three hectares</td><td>50</td></tr>
<tr><td>Over three hectares</td><td>75</td></tr>
<tr><td>Shops, Offices, Recreation Tourism</td><td colspan="2">Commercial developments of this type should have a water supply capable of delivering a minimum flow to the development site of between:</td><td>20 - 75</td></tr>
<tr><td rowspan="3">Education, Health And Community Facilities</td><td>Village and small community halls</td><td>Should have a water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 100m from the complex.</td><td>15</td></tr>
<tr><td>Primary schools and single storey health centres</td><td>Should have water supply capable of delivering a minimum flow of through any single hydrant on the development or within a vehicular distance of 70m from the complex.</td><td>20</td></tr>
<tr><td>Secondary schools, colleges, health and community facilities</td><td>Should have water supply capable of delivering a minimum flow through any single hydrant on the development or within a vehicular distance of 70m from the complex.</td><td>35</td></tr>
</table>

Source: National guidance document on provision of water for firefighting, Appendix 5, Water UK, May 2002

2.3.11 Non Revenue Water (NRW)

The International Water Association (IWA) has developed a detailed methodology to assess the various components of NRW. The manual user is encouraged to refer to the publication by International Water Association, Assessing NRW and its components - a practical approach, published on August 2003 for more detailed analysis. Accordingly, NRW has the following components:

- Unbilled authorized consumption
- Apparent losses (water theft and metering inaccuracies)
- Real losses (from transmission mains, storage facilities, distribution mains or service connections)

In many utilities, the exact breakdown of NRW components and sub-components is simply not known, making it difficult to decide the best course of action to reduce NRW. Metering of water use at the level of production (wells, bulk water supply), at key points in the distribution network and for consumers is essential to estimate levels of NRW.

NRW is sometimes also referred to as unaccounted-for water (UFW). While the two terms are similar, they are not identical, since non-revenue water includes authorized unbilled consumption (e.g. for firefighting and community/religious functions) while unaccounted-for water excludes it.

The most commonly used indicator to benchmark NRW is the percentage of NRW as a share of water produced. However, when losses in terms of absolute volume are constant, the percentage of NRW varies greatly with total water use, i.e. if water use increases and the volume of losses remains constant the percentage of NRW declines. This problem can be eliminated by measuring NRW not as a share, but in terms of absolute losses per connection per day, as recommended by the International Water Association.Also,losses per kilometer of network are more appropriate to benchmark real losses, while losses per connection are more appropriate to benchmark apparent losses.

There has not been a study specifically to estimate daily water losses in cubic meters per kilometer (Loss) of network in Uganda. However, such studies have been carried out in several parts of the world as indicated in Water in figures, DANVA's Benchmarking and Water Statistics 2010. This figure ranges from 1.5 in Netherlands, 42 in Brazil and 52 in China.In Uganda, the average UFW was 31% in 2005 (Mugisha, 2007). A correlation was derived in the countries with known UFW and losses per kilometer in cubic meters (Loss) of network as given below.

$$\boldsymbol{Loss = 64.522UFW - 2.249}$$

Using this relationship, it was established that in the case of Uganda, an allowance of 17 cubic meters per kilometer per dayshould be included to cater for leakage, wastage, water demand for the flushing of pipelines, storage tanks and others ordinary internal waterworks usage. If there is considerable leakage or unauthorized withdrawal of water from the distribution system, then the required supply of water will of course be much greater. In some cases, these water losses may rise to as high as 30 – 50. However, a detailed study should be carried out to establish this figure as it would give a clear indication of real loss.

2.3.12 Treatment and System Losses

Treatment water losses include water for backwashing filters, water lost during sedimentation, water used within the plant for cleaning filters, sedimentation tanks, etc. This water may be up to 10% of the raw water abstracted from the river or lake.

It should be noted that the average day unit water demand figures are presented as guidelines, which may be adjusted if different figures are shown to be more appropriate in a particular situation. The figures represent the mean values of water demands for the respective consumer types. For each given consumer type, there will be considerable variations in water demands from member to member.

2.4 Water Demand Patterns and Peak Factors

2.4.1 General

The daily water demand in a water supply scheme area will vary during the year due to seasonal climatic variations, work situations such as harvest seasons and other factors such as religious and cultural festivals. The figures given are average day unit demand figures, which are used in conjunction with the numbers of the domestic, institutional, commercial, industrial and other consumers to calculate the "Average Day Demand". However, the "Maximum Day Demand" used to design the capacities of the water sources, raw water transmission mains, treatment plants and treated water transmission mains, is estimated by adding 30% to the "Average Day Demand". Thus, the "Peak Day Factor" i.e. the quotient "Maximum Day Demand" / "Average Day Demand" becomes 1.3.

2.4.2 Rural Areas

In rural areas, it can be assumed that the bulk of the water used in a day is drawn between 7 a.m. and 7 p.m., but with hourly variations. Generally, two peak periods will be observed, one in the morning and the other in the evening. The same pattern can be assumed to apply for Private Connections and for Public Standpipes.

It is generally not economical, and not even technically feasible to design water sources, raw water transmission mains, treatment plants and treated water transmission mains to follow all such fluctuations in water demand, and thus to be able to meet those "Peak Hour Demands". This is the reason why water storage facilities are needed and provided as discussed in more detail in Chapter 9 – "Treated Water Storage".

However, distribution mains have to be designed with adequate capacity to meet the "Peak Hour Demands" of the consumers being supplied. Hence, the "Peak Hour Factor" i.e. the quotient "Peak Hour Demand" / "Maximum Day Demand" has to be determined for every scheme. It is this "Peak Hour Factor" which has to be multiplied with the "Maximum Day Demand" figures, in order to determine the "Peak Hour Demand" flows used in designing the capacities of the distribution mains. The "Peak Hour Factor" will depend on the size of the water supply scheme and the character of the community to be served.

"Peak Hour Factors" tend to be high for small rural communities and lower for larger communities and towns. Presently, there is little data available on diurnal demand pattern variations and "Peak Hour Factors" in rural areas, but "Peak Hour Factors" given in Table 2-8 can be used as guidelines.

Table 2-9: "Peak Hour Factors" for Rural Areas

Population [Pe]	Peak Hour Factors
1,000 or more	2.0
500	2.5
200	3.0
100	3.5
50	4.5

Linear interpolation may be used to determine "Peak Hour Factors" for intermediate population figures. Where reliable records of actual consumption patterns do exist, corresponding "Peak Hour Factors" should be calculated accordingly. Large institutions and industries may have their own on-site balancing storage facilities which may somewhat attenuate "Peak Hour Demands" in distribution systems. Such storage facilities should be encouraged and considered when determining the design flows and capacities of the distribution mains. With EPANET, typical demand curves are given in the charts below. The patterns may be applied to supply areas or even to taps, depending on the size of the network

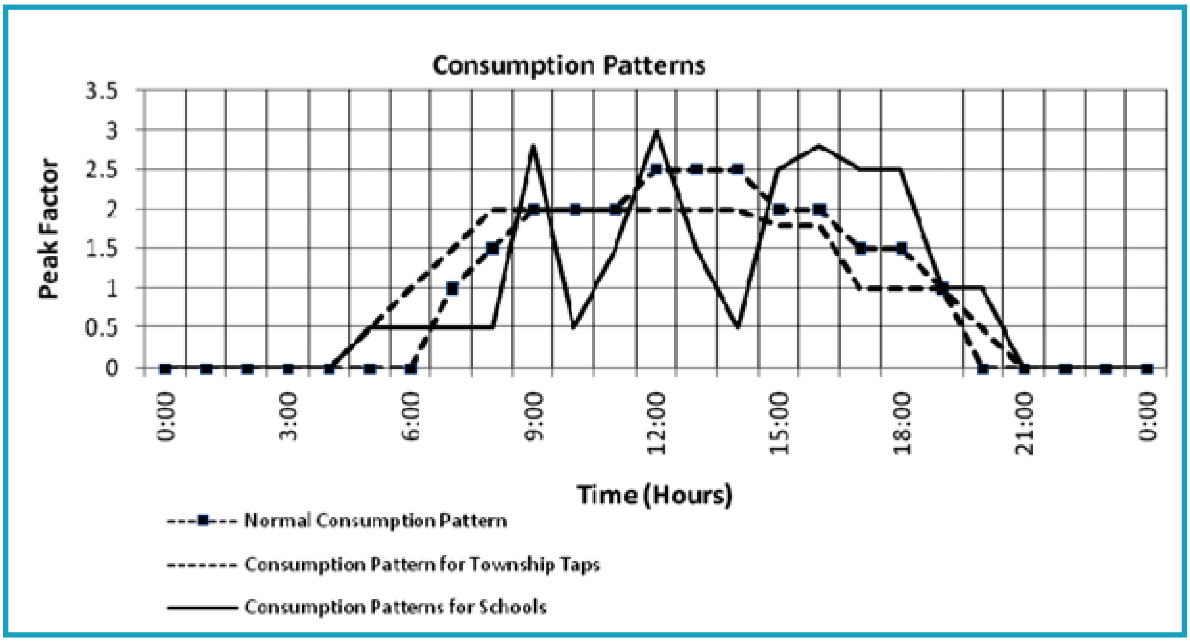


Figure 2-2: Typical Demand Patterns For Various Users.

2.4.3 Urban Centres

Demand pattern and "Peak Hour Factors" adopted in the design of water supply schemes in large towns and urban centres should be based on the analyses of records from the existing water supply systems. If such information is not available, records from other similar towns may be used.

Chapter 3

WATER SOURCES

3.1 General

3.1.1 General Selection Considerations

The selection of a suitable source or combination of sources of water is one of the initial steps in designing a water supply scheme. The source or sources must be capable of supplying sufficient water of acceptable quality for the scheme.

Water sources can be broadly divided into surface water sources and groundwater sources. The selection of the most suitable water source involves taking into account a number of general factors as follows:

- **i)** **Quantity**: Is the quantity of water available at the source sufficient to meet present and future demand? Water extraction permits from the Directorate of Water Resources Management in Entebbe are required for water abstraction in Uganda.
- **ii)** **Quality**: Is the raw water quality such that, water which meets the quality standards specified in Chapter 5. Drinking water sources must meet the minimum WHO water quality standards.
- **iii)** **Cost**: Are the capital as well as the operation and maintenance costs of the source acceptable?
- **iv)** **Technology**: Is there appropriate technology and expertise to exploit and maintain the source of water and associated water treatment and transmission facilities?
- **v)** **Protection**: Can the water source be protected from present and future pollution and contamination and can the catchment area be protected effectively to ensure the sustainability of the quantity and quality of the raw water?

The freshwater sources of Uganda include surface water (rivers, streams and swamps), ground water (deep and shallow wells, springs) open water bodies (lakes) and rainfall. Figure 3-1 and Figure 3-2 show the general distribution of rainfall and ground water resources in Uganda.

3.1.2 Specific Selection Considerations

Sources which require little or no treatment of raw water such as springs, wells and boreholes should be given the highest selection priority provided their yields are sufficient to meet the water demands of the scheme.In selecting surface water sources, rivers with upland mostly forested catchments should be given preference.

Sources from which water can be supplied by a gravity system are particularly more favourable than those which require pumping. For household and small community water supplies, rainwater harvesting will serve quite well in most areas of Uganda.

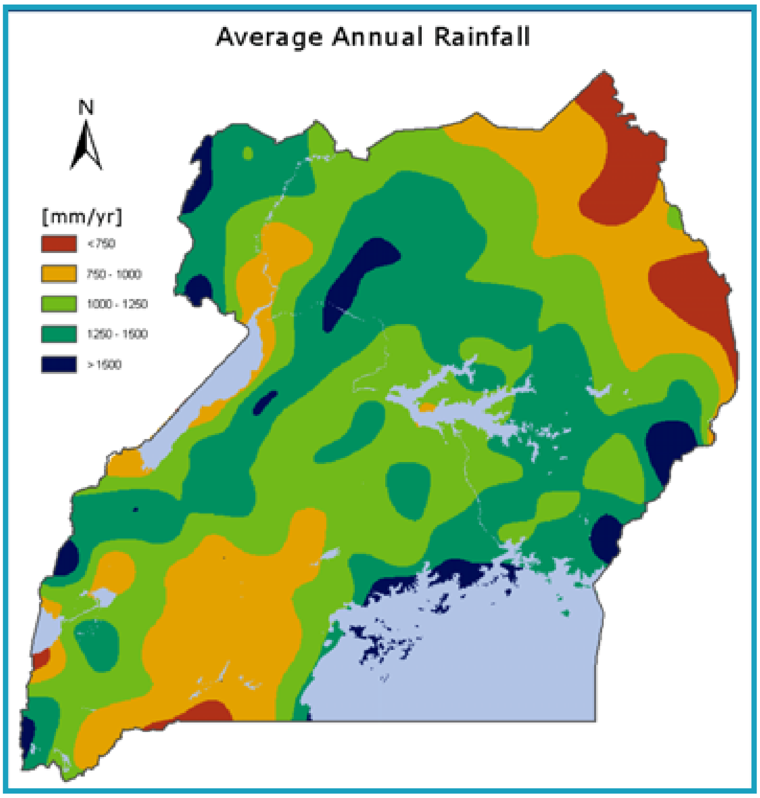


Figure 3-1: Annual Average Rainfall in Uganda
Source: MWE, National Water Resources Assessment Report, 2011

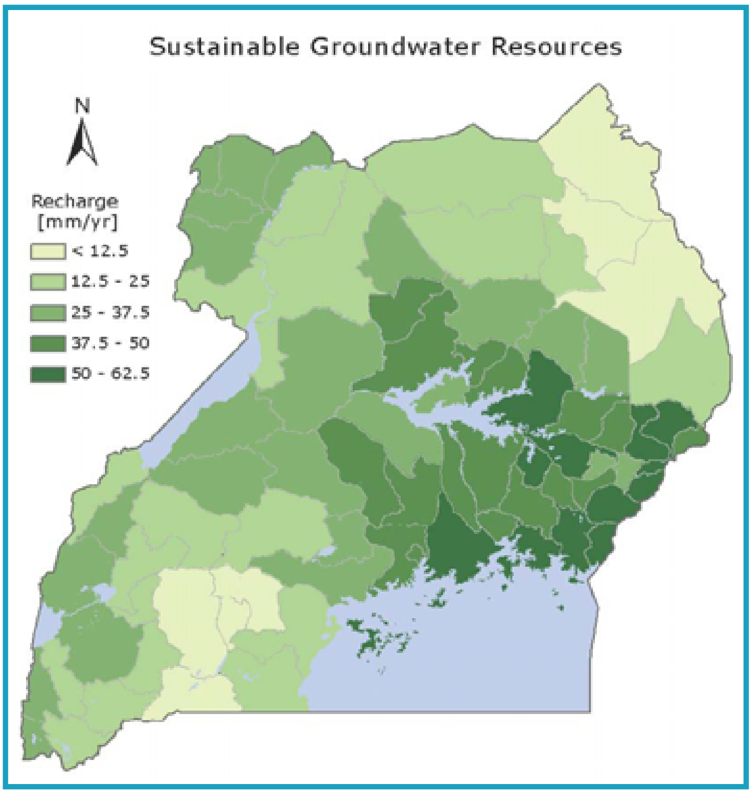


Figure 3-2: Ground Water Resources in Uganda
Source: MWE, National Water Resources Assessment Report, 2011.

Table 3-1: Renewable Water Resources in Uganda (1953-1978 Timeframe)

River basin	Outflow location	Total Area	Rainfall	Runoff	Change in lake storage	Actual Evapotrans-piration	Potential Evapotrans-piration	Runoff coefficient	EA/ Ep	Runoff
		A	P	Q	S	Ea	Ep			Q
		(km^2)	(mm)	(mm)	(mm)	(mm)	(mm)			(bcm/yr)
Lake Victoria	Lake Victoria outlet	252,800	1,264	130	4.9	1,129	1,453	0.11	0.78	32.86
Lake Kyoga	Lake Kyoga outlet up-stream Kafu confluence	57,236	1,132	22	4.6	1,105	1,698	0.02	0.65	1.26
Victoria Nile	Inflow to Lake Albert from Kyoga Nile	27,961	1,253	52	0	1,201	1,547	0.04	0.78	1.45
Lake Edward	Lake Edward outlet	26,719	1,144	152.4	-0.1	991.7	1,281	0.13	0.77	4.07
Lake Albert	Lake Albert outlet	31,496	1,231	-177	13	1,335	1,452	-0.095	0.91	-3.69
Aswa	Outflow from Aswa (Downstream of stations 86202 and 86212)	27,637	1,212	62	0	1,150	1,715	0.05	0.67	1.71
Albert Nile	Outflow from Albert Nile (downstream of 87221 and 872017)	20,727	1,274	-12	0	1,286	1,572	-0.01	0.81	-0.25
Kidepo	Not included in model	3,229	1,112				1,749			
Miscellaneous	Not included in model	5,716								
Total										37.41

Source: MWE, National Water Resources Assessment Report, 2011

3.2 Rivers and Streams

3.2.1 Introduction

Uganda's rivers and lakes, including wetlands, cover about 18% of the total surface area of the country, with rainfall being the greatest contributor to the surface and ground water resources. Almost the whole of Uganda lies within the Nile basin, which is shared by 10 countries. The most significant hydrological feature in Uganda is Lake Victoria, the second largest freshwater lake in the world, which is also the source of the Nile, the longest river in the world (Uganda National Water Development Report, 2005).

Table 3-2: Major Rivers in Uganda.

Basin Area, km^2	Annual Average	Yield, mm	Mean Flow (m^3/s)
Victoria Nile	57,669	133.05	1,120.35
Kyoga Nile	26,796	98.57	1,051.73
Edward and George	18,624	211.95	159.14
Aswa	26,868	102.01	42.91
Albert Nile	20,004	96.45	1,262.45

Source: MWE, National Water Resources Assessment Report, 1998

Table 3-3: Major Lakes of Uganda

Lakes	Total area (km^2)	Area in Uganda (km^2)	Height above sea level(m)	Catchment Area (km^2)	Catchment area in Uganda (km^2)	Maximum Depth (m)
Victoria	**68,457**	**28,665**	**1,123**	**184,000**	**59,858**	**82**
Albert	**5,335**	**2,913**	**621**	**n.a**	**18,223**	**51**
Edward	**2,203**	**645**	**913**	**12,096**	**18,624**	**117**
Kyoga & Kwania	**2,047**	**2,047**	**1,033**	**75,000**	**59,669**	**7**
Salisbury (Bisina)	**308**	**308**	**1,047**	**n.a**	**n.a**	**n.a.**
George	**246**	**246**	**914**	**9,705**	**n.a**	**3**

Source: MWE, State of the Environment Report, 2002

3.2.2 Environmental Residual Flows

Environmental flows describe the quantity, timing, and quality of water flows required to sustain freshwater and estuarine ecosystems and the human livelihoods and well being that depend on these ecosystems(**The Brisbane Declaration, 2007)**. Through design and implementation of environmental flows, it is required to achieve a flow regime, or pattern, that provides for human uses and maintains the essential processes required to support a healthy river ecosystems. Environmental flows do not necessarily require restoring the natural, pristine flow patterns that would occur absent human development, use, and diversion but, instead, are intended to produce a broader set of values and benefits from rivers than from management focused strictly on water supply, energy, recreation, or flood control.

Environmental flow components, or events, may be quantified in terms of magnitude (the volumetric flow rate or level in m^3/s), timing when the low flow event occurs (the dry month), duration (how long the low flow event lasts in days), frequency of occurance of the low flow event (return period) and rate of change of low flow over time (m/day of flow recession). The recommended Environmental Residual Flow varies for individual rivers and streams and therefore a comprehensive Environmental Impact assessment should be carried out and approved by NEMA to determine this flow.

3.2.3 Safe Yield

Safe yield is the term used to express the amount of water an aquifer or well can yield for consumption without producing negative effects to the aquifer and the environment. For a river/stream, safe yield represents the minimum flow rate that will guarantee no risk to the river hydrology and its surroundings. Safe yield is estimated so as to check whether the planned withdrawal for water supply purposes will be safeguarded in the long run. Generally the amount of water withdrawn should not exceed the natural recharge. Potential effects are contamination of the aquifer water by induced infiltration, decreased river flows, and lowering of the water table.

To determine the "Safe Yield" of a river or stream, a flow – frequency/probability analysis should be performed and plotted on appropriate statistical graph using the lowest recorded daily flow of each calendar year for which records are available for the dry seasons. From this analysis and graphs, the 95% probability of daily low flow should be determined and taken as the "Safe Yield" of the river or stream. Surface water monitoring network in Uganda is shown in Appendix 2a.

3.2.4 Flood Flows

Small dams, (with a height less than 4 m), spillways and intake structures should be designed for the respective 100-year return period floods.

3.2.5 Flow Records

When dealing with a river or stream with no or few flow observation records, full use should be made of flow records and rainfall data from adjacent rivers or similar catchments to construct a probable flow-frequency/probability curve. Rivers and streams which are identified as sources of water for water supply development should be installed with flow and level measuring devices as part of the project development. A procedure should be provided for the taking and keeping of records from the flow and level measuring devices. This procedure is available from the Directorate of Water Resources Management.

For design purposes estimation of stream and river flow requires several routine measurements along the stream. Accuracy of the stream flow data will depend on the physical features of the cross section, the instrument being used and the frequency of measurement. Stream flow data for design purposes should be collected at intervals, say monthly on a continuous basis. This is preliminary real time data that can be accessed by users on the internet if uploaded. The mean discharges of the day and extremes are then computed. This is the final data that can be reliably used for design purposes. The user is referred to the Manual on Stream Gauging, 2010, Vol. 1 & 2 by the World Meteorological Organisation for guidance on stream and River Gauging.

3.2.3 Ungauged Rivers

Data availabilityand quality especially for river flows in Uganda has always been a challenge. In case of missing data, statistical gap filling can be employed, however, in case of un gauged rivers, we propose a five -step prediction to be used as follows:

i) Identify publicly available input data based on traditional ground measurement inside the catchment area.

ii) Collect river flow data for a period not less than 3 years. This data is then split into 3 part to be used for warming up, calibration and validation.
iii) Select appropriate hydrological model based on temporal resolution of data
iv) Calibrate values of the model parameters using ground measurement such that the model closely simulates the hydrological behavior of river.
v) Validate the model using the last part of the records and compute the efficiency. This model can then be used for prediction and forecasting for the un-gauged rivers.

3.3 Springs

3.3.1 General

Springs are found mainly in mountainous or hilly areas. A spring can be defined as a place where rock or clay layers block the flow of underground water, forcing it upwards where the outflow emerges in the open at the ground surface.

In Uganda, there are two main types of springs as follows:

i) Gravity springs: occurring in unconfined aquifers; and
ii) Artesian springs: occurring in aquifers overlain by confining impervious layers preventing the water from rising to its free water table level, and therefore kept under pressure.

To locate good springs and to get information about their reliability especially during drought periods the designer should consult the local people resident in the area. For springs with high yields that may be considered for high demand pipe schemes, there should be a provision for flow measurement to monitor the variation of the spring yield over time. It is common to find springs without any flow records.

3.3.2 Yield

To measure the yield of springs identified as potential sources for a water supply scheme, simple devices such as over-flow weirs and V-notches should be installed as early as possible in the planning process of the scheme. Spring yield is measured in litres per second (l/s). The measurement process involves two selected trained people who measure the discharge from the spring over the study period.

The process starts with the construction of an earth dam. Spring water retained by the dam is drained through a pipe. One person collects the water with a container of a known volume while the other measures the time needed to fill the container. The pipe diameter and container size are chosen such that the water outflow will not fill the measuring container in less than five seconds. Sometimes several pipes are used. Four readings are taken during the day and day averages are calculated, expressing the discharge in l/s. This is repeated once every week for the measuring period. In this way, the minimum and maximum yields are determined. Measurement of yield should be done both in the dry and wet seasons, for determination of safe yield, the dry season yield is particularly important. In all developments on springs, there shall be installed a flow measuring device to monitor flow rate for springs over time.

Studies of catchments with many springs channelled into a single supply point must be carried out carefully for reasons of back-pressure effects. Simply adding the yields from each individual spring together is not enough. An excessive flow could build up a back pressure and cause some springs to divert their courses. In some cases it may lead to permanent damage to the catchment. The designer needs to study the flow characteristics of any collection chambers or pipes and ensure that each spring outlet flows freely.

Flows from artesian springs often fluctuate less than flows from gravity springs. Variations in flow of gravity springs can be considerable and therefore many measurements are required to determine if the spring can supply sufficient water for a planned scheme. The bacteriological quality of water from artesian springs also tends to be better, because the impervious confining layers protect the water in the spring aquifer against contamination. Therefore in most cases artesian springs are to be preferred to gravity springs. For water supply designs purposes the safe yield of springs should be estimated as 2/3 of the dry season yield. Figure 3-3 shows the distribution of springs in Uganda. This serves as guide on where spring protection may be considered as a water supply option.

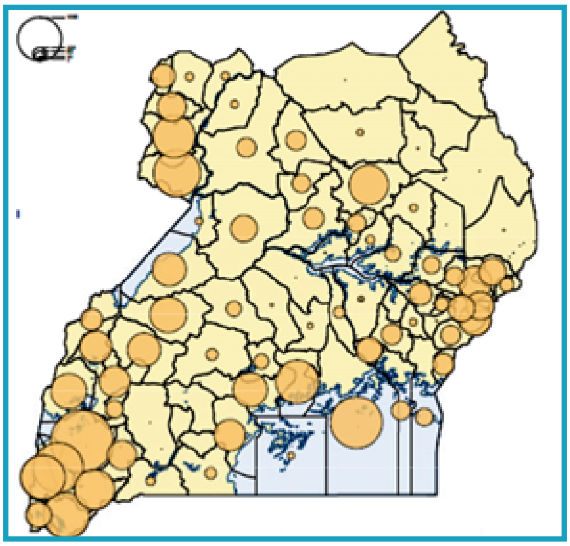


Figure 3-2: Spring Distribution in Uganda.
Source: Uganda Water Supply Atlas, 2010

3.3.3 Spring Protection

Springs should be protected to prevent contamination by surface water. The ground acts as a bacterial filter making spring water a reliable water source. To protect a spring the following steps are followed:

i) Clean up the whole site by digging drainage trenches;
ii) Place a layer of hardcore over which is an impervious clay layer; The spring water is collected and channeled to the discharge pipe in a concrete wall through a gravel layer;
iii) Spring water is collected and channelled through a gravel layer to the discharge pipe in the concrete wall. The pipe is located at a convenient height to enable villagers to fill their containers;
iv) An impervious clay layer is used above and around the spring to restrict surface seepage. A drainage channel is dug to channel away storm water and a concrete paved access provided to enable users to easily fill containers; and
v) A fence may be built to keep livestock out and the grass surrounding the spring kept well-trimmed.

Common materials are used in the construction of springs: Stones, aggregates and sand are obtained locally and cement used ordinary Portland cement. The walls may also be built from local stone by skilled "Fundis" minimizing the use of cement hence lowering the cost. Further information on spring protection and construction can be obtained from the Water Aid website: http://www.wateraid.org/uk/what_we_do/sustainable_technologies/technology_ notes/245.asp

3.4 Boreholes and Wells

3.4.1 Basic Considerations

The predominant water supply technology used in Uganda is the deep borehole. Approximately 38% of the population with access to safe water supplies is served by deep boreholes. This is followed by protected springs (26%) and shallow wells (25%). The large number of boreholes is suggestive of the reliability of ground water resources and the appropriateness of the borehole technology as a means of abstracting ground water. Figure 3-4 gives a guide on where borehole technology can be considered as a water supply option.

The potential of shallow wells is quite high, especially in the valleys. Their potential is favoured by the thick regolith that is fairly coarse grained. From Uganda's experience, shallow wells are a very reliable source of water supply to the communities although precautions need to be taken to ensure that they are not contaminated. Susceptibility to contamination is the major reason why the shallow well is becoming an unpopular technology.

The safe or long-term yield of a borehole or well can be defined as the maximum quantity of water that can be obtained permanently from the borehole or well. The safe yield has to be estimated to see whether the planned abstractions for water supply purposes can be sustained in the long term.

The long-term yield evaluation of a water supply borehole relies on the following factors:

i) Estimations of recharge;
ii) Calculation of hydrogeological parameters such as Transmissivity (T), Storage Coefficient (S), Skin Factor and others; and
iii) Analysis of aquifer boundary conditions.

For proper aquifer analysis, test pumping periods should be extended to 1 week. This is important especially in major water supply schemes where large capital investments are involved. However, experience in Uganda over the last 20 years shows that pumping tests of at least 72 hours can provide aquifer characteristics information that can guide design.

A 72-hour test should be carried out for a new water supply borehole, to facilitate the choice of pump capacity. Such a test will not provide sufficient information on aquifer yield. However, the aquifer type and properties must be identified and understood if serious mistakes are to be avoided.

Estimation of aquifer boundary conditions is a major problem after a short pumping test. In such cases, the judgment of a skilled hydro-geologist will be required. Negative hydraulic boundaries within a given recharge area will reduce yield, whereas positive boundaries will increase yield.

The recharge rate and the estimated recharge area of the aquifer are the other important parameters required to estimate long- term yield. These parameters are best evaluated based on long-term monitoring of borehole abstractions, groundwater levels and rainfall since recharge may not take place every year, the pump capacity in a new borehole should be chosen assuming that recharge may not occur for periods of up to 5 years.There are several observation wells around the country that are monitored by the Directorate of Water Resources management. Records of these should be consulted to provide additional information on ground water characteristics in Uganda.

3.4.2 Borehole Capacity Evaluation

3.4.2.1 Test Pumping

For long-term planning, it is very important to obtain accurate production capacity data for each borehole. However, for practical reasons, routine test pumping should be limited to 48 hours for hand pumped wells and 72 hours for motor production wells. The aim of the pumping test is to determine a reasonable capacity for the production pump. However in semi-arid climates the capacity can be easily over estimated if complete aquifer analysis is not done. Therefore, it is important to compare the test results to the estimated storage capacity and the long-term recharge within the area of influence of the borehole.The pump used for test pumping should have adequate capacity to give maximum information without pumping the borehole dry prematurely.

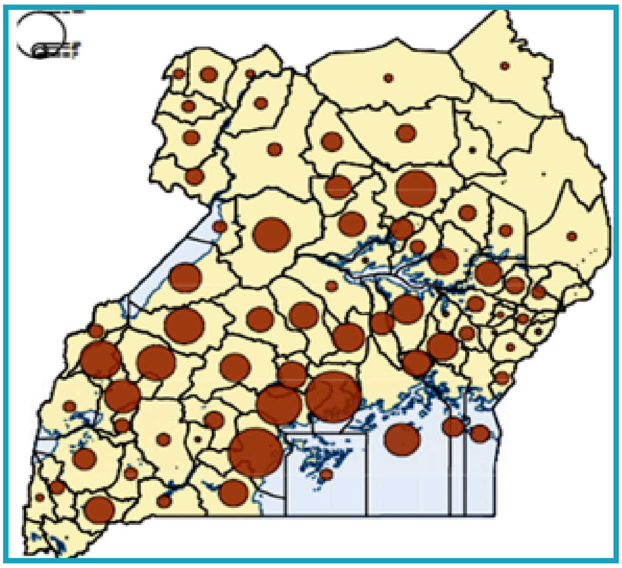


Figure 3-3 Borehole Distribution in Uganda
Source: Uganda Water Supply Atlas, 2010

The three main tests to be carried out on bore holes are:

i) Step test – designed to determine the short term relationship between the yield and the draw - down of the borehole being tested;
ii) Constant rate test – pumping is carried out at constant rate for a longer duration than the step test and is used to provide information on the hydraulic properties of the aquifer; and
iii) Recovery test – it is carried out by monitoring the recovery of the water level after pumping has stopped at the end of the constant rate test or step test. It is used as a check on aquifer characteristics determined by other tests.

Table 3-4 gives a summary of different borehole water uses and the corresponding tests for each.

Table 3-4: Recommended Test and Duration to Estimate Sub-Surface Water Yield

Use Of Water	Test	Duration	Recovery Test
Stock or domestic	Extended step	Total 6 hours	Up to 3 hours
Hand pump	Extended step	Total 6 hours	Up to 3 hours
Town water supply Low-yield borehole	Step Constant discharge	4 x 1 hour 24 hours	- Complete
Town water supply High-yield or main borehole	Step Constant discharge	4 x 1 hour 72 hours or more	- Complete

Sources: Guidelines for Human Settlement Planning and Design Vol. 2, ICRC 2005.

For further reference, detailed descriptions of each of these tests and analysis of data can be obtained from the ICRC Practical guidelines for Test Pumping in Water Wells - Technical Review 2011.

3.4.2.2 Long Term Test-Pumping

A longer term test pumping of upto 1 week is necessary to ascertain the water quality and to ensure that the long-term yield is sustainable. An extended period of pumping will also ensure that the borehole is adequately cleared of drilling cuttings.

The test pumping procedure should include the following tests:

i) An airlift test carried out on completion of the borehole, to provide information for the selection of the appropriate pumps for the following tests;
ii) A 1-hour test with accurate recording of drawdown and capacity, to facilitate the estimation of the capacity for the following 72 hour test; and
iii) A 72-hour test, to facilitate the estimation of the production capacity.

3.4.2.3 Evaluation of Test Pumping Data

A practical way to make a reasonably reliable evaluation of the test pumping data is to use Jacob's Extrapolation Method or other similar methods where the drawdown is plotted against the logarithm of the pumping time. The 5-year capacity and drawdown are found by extrapolation on the plotted graph, to give an estimate of the long-term yield of the borehole. The production pump capacity can then be calculated using the appropriate formulae incorporating the pump level below the static water level, the capacity for the 48-hour test and the 5-year drawdown as extrapolated on the "Jacob's graph".

3.4.2.4 Recovery

When pumping is stopped, the water level in the borehole and aquifer will rise towards its pre-pumping level, the static head. The rate of recovery can yield useful information about the present aquifer conditions in a similar manner to the pumping test. Therefore, recovery analysis and evaluation should be done by an experienced Hydro-geologist in order to crosscheck the calculations based on test pumping.

3.4.2.6Existing Borehole Data

Data on yields from existing boreholes should be used with caution. After years of use of the boreholes the yields might have changed or what is recorded as the yield may actually be the capacity of the pump used for the test pumping. If the background of the reported yield of a borehole is suspect, fresh test pumping is recommended to be done.

3.4.2.6 Well Data

A file should be established on each supply at the time when plans for its construction are initiated. From initial planning to the final abandonment of the well, the following records should be generated and carefully preserved in this file as follows

i) Initial design file should include drawings or written specifications on diameter, proposed total depth, position of screens or open hole, method of construction and materials to be used I the construction;
ii) Construction record file should include the method of construction and the driller's log, geophysical log of the materials penetrated during construction, the diameter of the casings and screens, the depths of casings and screens, total depth of well and the weight of the casing, records of all logs for all test wells including those that were not successful due to small yields;
iii) Well acceptance test file should include a copy of the water level measurements made before, during and after the drawdown test, a record of the pumping rate, copies of graphs and data and copy of the hydrologist's report on the interpretation of the test results;
iv) Pump and installation file should include pump type, motor specifications, depth to pump intake, a copy of the pump manufacturers performance and efficiency data and data on the length of the air line or a description of facilities provided for water level measurements including a description of the measuring point;
v) Operation record file should include data on the type of meter used to measure the flow rate, weekly measurement of the static and pumping water levels and periodic analysis of water quality;

vi) Record of well maintenance file should include the dates and the activities instituted to increase the yield or to improve the water quality and the data showing the results achieved; and
vii) Record of well abandonment file should include the date that the use of the well was discontinued and the description of the methods and materials used to seal or plug the well.

3.4.3 Aquifers

An Aquifer is a geologic formation that contains sufficient saturated permeable material to yield significant quantities of water to a well. Aquifers can be divided into three main types as follows:

i) porous like in sand rivers;
ii) fractured like in crystalline bedrock; and
iii) fractured/porous like in sedimentary rocks, limestone, sandstones, etc.

Either porous or fractured systems may be dominant in a given area depending on the local conditions prevailing in the area. An aquifer is an underground layer of water-bearing permeable rock or unconsolidated materials (gravel, sand, silt, or clay) from which groundwater can be usefully extracted using well. Aquifers are typically saturated regions of the subsurface and can occur at various depths. Aquifers can be porous like in sand rivers, fractured like in crystalline bedrock or fractured and porous like in sedimentary rocks for example limestone and sandstones. Either porous or fractured systems may be dominant in a given area depending on the local conditions prevailing in the area.

Typically, aquifers are classified according to their boundary conditions including confined and unconfined aquifers. Unconfined aquifers also referred to as phreatic aquifers have their upper boundary is the water table or phreatic surface. Typically, the shallowest aquifer at a given location is unconfined, meaning it does not have a confining layer (an aquitard or aquiclude) between it and the surface. Unconfined aquifers usually receive recharge water directly from the surface, from precipitation or from a body of surface water (e.g., a river, stream, or lake) which is in hydraulic connection with it.

Confined aquifers have the water table above their upper boundary (an aquitard or aquiclude), and are typically found below unconfined aquifers. The water here completely fills an aquifer that is overlain by a confining bed and the water in this aquifer is said to be confined. These aquifers are sometimes referred to as artesian aquifers.There are a number of basic hydrogeological differences between porous aquifers and fractured aquifers as outlined in Table 3-5.

Table 3-5: Differences Between Porous and Fractured Aquifers

Porous Aquifers	Fractured Aquifers
Normally horizontal bodies	Normally vertical or sub vertical features
High storage capacities	Low storage capacities
High regional ground water “transport capacity”	Low regional “groundwater transport” Capacity

3.4.4 Estimation of Potential and Natural Recharge

Different methods for measuring natural recharge exist and a few of them include:

i) Physical methods – they rely on physical measurements of hydrological parameters or of aquifer and soil physical parameters. Physical methods can both Direct and Indirect physical methods. These methods are inexpensive and quick but have drawbacks because small physical changes in some parameters are undetectable by physical methods, several measurements are required for some sites due to variations in topography, soils and vegetation and extreme temporal variability in arid areas will require long time series to measure the mean annual recharge;

ii) Methods based on saturated and unsaturated zones;
iii) Chemical and Isotopic methods;
iv) Methods based on analysis of inflow, outflow and aquifer response; and
v) Methods based on the modeling of ground water flow, soil water flow or plotting of the hydrological balance at the field or watershed scales.

The methods available for the estimation of ground water recharge directly from precipitation can be broadly divided into three-inflow, aquifer response and outflow methods according to studies conducted by Kumar C.P, 1977. The following methods are commonly in use for estimating natural ground water recharge as presented by Sathish Chandra, 1979.

i) Soil water balance method
ii) Zero flux plane method
iii) One-dimensional soil water flow model
iv) Inverse modelling technique
v) Ground water level fluctuation method
vi) Hybrid water fluctuation method
vii) Ground water balance method
viii) Isotope and solute profile techniques

For detailed estimation of potential and natural recharge, the user could review these studies.

3.5 Sub-Surface Sources

3.5.1 General

In seasonal rivers, it is often possible to abstract water from the river bed during the dry season if a suitable structure is built across the river bed under the surface to impound the sub-surface flow. The water is then collected through infiltration galleries constructed upstream of the sub-surface dam. A typical infiltration gallery is illustrated in Figure 3-5.

3.5.2 Water Abstraction

River beds can sometimes store considerable amounts of water which can be extracted during the dry seasons. The river beds are then recharged during the rainy seasons. The quantity of water available in river beds can be estimated using the specific yield of the different aquifer materials as explained elsewhere in this manual. The geometry and hydraulic properties of a river bed should be carefully investigated before accurate estimations of the available water can be done.

It is sometimes possible to find a natural barrier in form of a rock outcrop or impervious material under the river bed. Large amounts of water can be found stored behind such barriers. The total sub-surface water flow under a river bed can be estimated by use of Darcy's law. Darcy's Law is a relationship that explains fluid flow in porous media, such subsurface water flow. The Mathematical relationship which is used to describe the flow of ground water through an aquifer is known as the Ground water flow equation. The basic formula to calculate the flow rate, Q, is:

$$Q = \frac{KA(h_1 - h_2)}{L}$$

Equation 3-1

Where:
- K is a permeability coefficient (describes the porosity of the underground formation),
- A is the cross sectional area,
- h_1 is the height of the inlet head,
- h_2 is the height of the outlet head, and
- L is the path length of the flow.

In practice, not more than 75% of the sub-surface flow can be impounded by a subsurface structure.

3.5.3 Artificial Recharge

3.5.3.1 General

Under suitable conditions, it is possible to supplement the natural recharge of an aquifer and thus add to its safe yield capacity. This is called artificial recharge. Artificial recharge entails measures to feed water from surface sources such as rivers and lakes into the aquifer, either directly or by spreading the water over the infiltration area and allowing it to percolate downward into the aquifer. Apart from adding to the yield of the aquifer, artificial recharge also provides purification of the infiltrated water. Artificial recharge should be considered in cases where soil conditions are suitable, and where the quality of the original water source is unacceptable and/or where the quantity is insufficient. In most cases, groundwater is recharged with surface water. Occasionally, high yield, poor quality groundwater sources are used to supplement low yield, high quality groundwater sources.

Ground waters are usually superior to surface waters in terms of bacteriological, physical and chemical quality. Where groundwater yields are inadequate, the feasibility of supplementing them by artificial recharge from adjacent surface water sources should be investigated. Spreading and well recharge are common methods of artificial recharge.

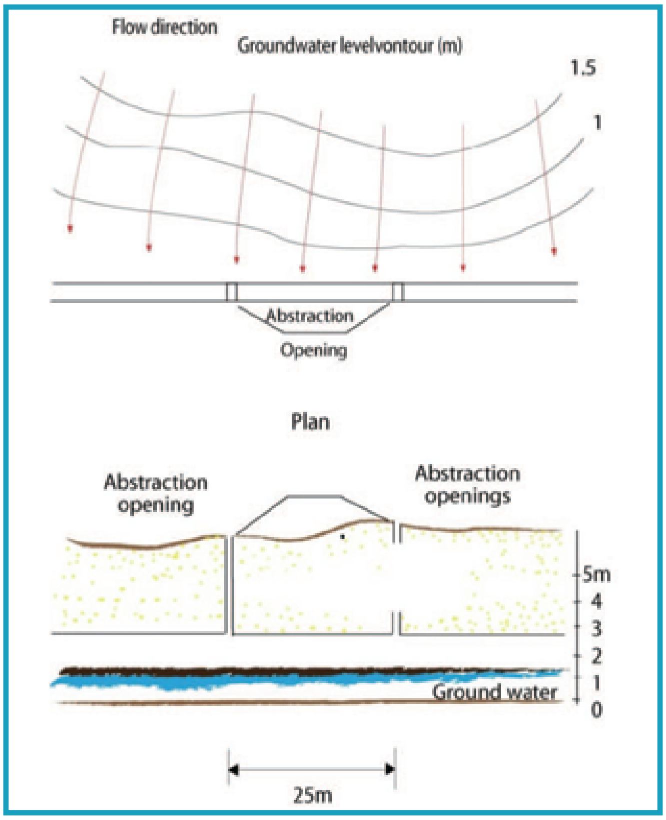


Figure 3-4: A Typical Infiltration Gallery.
Source: www.unep.or.jp [accessed on February 24, 2012]

3.5.3.2 Spreading Methods

The most favourable hydrogeological conditions for water spreading are where shallow aquifers and permeable soils which exist in areas with quaternary sedimentary rocks (unconsolidated sediments). However, favourable conditions may also exist in parts of the other geological areas, depending on local conditions.

3.5.3.3 Well Recharge

The most favourable hydrogeological conditions for well recharge are where deep permeable aquifers and less permeable soils exist in areas with sedimentary or volcanic rocks.

3.5.3.4 Bank Infiltration

Bank infiltration is one of the principal methods of artificial recharge of aquifers. It involves the construction of galleries or lines of wells parallel to the shoreline of a river or lake, at a sufficient distance as illustrated in the figure below.

In the original situation, the outflow of groundwater feeds the flow of the river or lake. The withdrawal of groundwater at high abstraction rates will lower the groundwater table near the shoreline below the water level in the river. In this situation, the abstracted water will, for the most part, be induced recharge, that is water originating from the river.

In order to provide for sufficient time for the purification of the water during its flow from the river or lake to the gallery or line of wells, the distance X should not be less than 20 m and preferably more than 50 m. However, often it may not be practicable to place the recovery point so far away from the stream or lake because of the local ground conditions. In this case, provisions should be made for disinfecting the water.

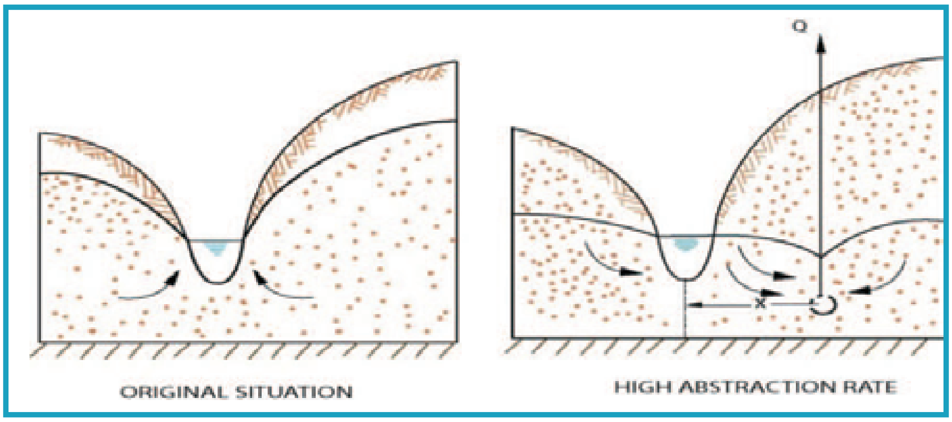


Figure 3-5: Bank Infiltration

The most favourable conditions for bank infiltration are where shallow aquifers and permeable soils exist in areas with quaternary sedimentary rocks (unconsolidated sediments). The identified area should be combined with a map on surface water availability to determine areas where bank infiltration is really feasible. As a result the total area suitable for bank infiltration will be smaller.

3.5.3.5 Storage Dams

The most favourable hydrogeological conditions for sand storage dams exist in areas with shallow aquifers and permeable soils, which are generally found in areas with quaternary sedimentary rocks (unconsolidated sediments). However sand dams may also be feasible in small river valleys in other geological areas with a thin alluvial or weathered zone, which are too small to be represented on the geological map.

The two techniques with the largest suitable area are water spreading and sand dams. The identified areas correspond with gently sloping areas with medium to coarse soils and arid to dry climatic conditions. These are found in the savannah areas south of the Sahara and in eastern and southern Africa. Table 3-6 is a section from a table of a summary of artificial recharge methods and their applicability in different countries.

Table 3-6: Applicability of Different Artificial Recharge Methods

Country	Area (Km^2)					% Area			
	Total	Water Spread	Well Recharge	Bank Infiltration	Sand Dam	Water Spread	Well Recharge	Bank Infiltration	Sand Dam
Uganda	267,602	38,813	-	122,861	69,417	15%	-	46%	26%

Source: MARS Global Opportunities, 2004.

3.5.4 Artificial Aquifers

Artificial aquifers should be designed for a retention time of water underground of 60 days. For shorter retention periods, disinfection of the water will be necessary.

Typical specific yields or storage capacities of different aquifer materials are as shown in Table 3-7. It must be appreciated that large variations in the given values are to be expected and that careful investigations will be required to determine actual specific yields values accurately.

Table 3-7: Specific Yields of Different Aquifer Materials

Material	Specific Yield (%)
Gravel, coarse	21
Gravel, medium	24
Gravel, fine	28
Sand, coarse	30
Sand, medium	32
Sand, fine	33
Silt	20
Clay	6
Sandstone, fine grained	21
Sandstone, medium grained	27
Limestone	14
Dune sand	38
Loess	18
Peat	44
Schist	26
Siltstone	12

Source: www.aqtesolv.com/forum/properties [accessed on March 05, 2012]

3.5.5 Sand Dams

A sand dam is a reinforced concrete wall built in a seasonal riverbed to capture and store water beneath the sand. The captured water is filtered and protected by the sand and the dam. Seasonal rains quickly fill the dam with water and soil which is made up of both sand and silt. The heavier sand sinks and builds up behind the dam while the lighter silt washes downstream. The sand accumulates behind the dam until the dam is completely full of sand up to the spillway. Water is stored within the sand making up 25 – 40% of the total volume. The actual volume depends on the sand particle size and the size of the dam. Water from sand dams can be abstracted by scooping holes, pipe filtration or shallow wells with pumps. Because the water is stored below the sand, evaporation is reduced. Figure 3-7 shows a typical sand dam construction.

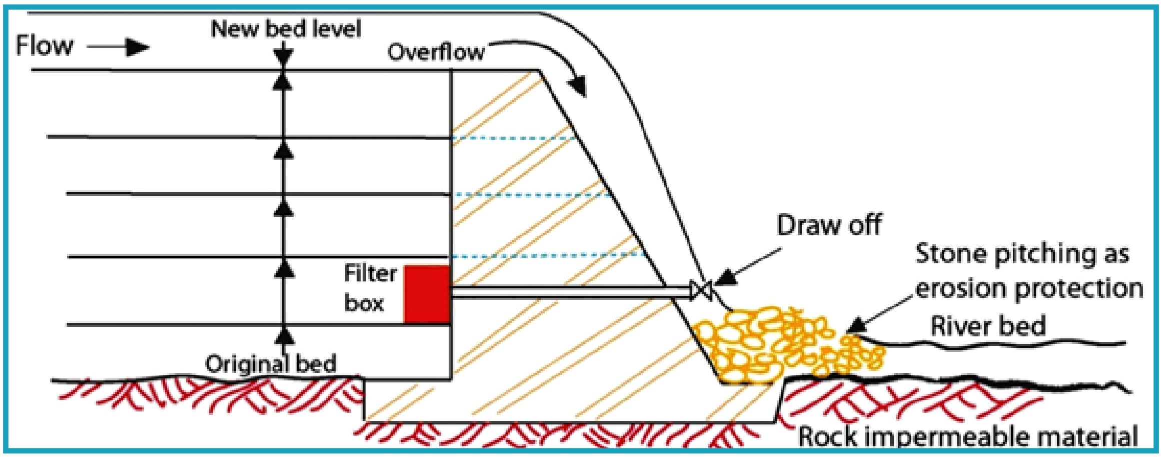


Figure 3-6: A Sand Dam.
Source: Water Aid, 2000.

3.6 Rainwater Harvesting

3.6.1 Rainfall Data

Rainwater harvesting entails the utilization of rain falling on house roofs, natural ground, roads, yards or other specially prepared catchment areas. Rainfall over Uganda varies quite considerably, between different seasons and between different parts of the country. Figure 3-1 gives average annual rainfall in Uganda and can be used to estimate rainwater for a given area. Additional relevant information of Rainfall distribution in Uganda is given in Appendix 3D**.** This information can be used to make rough estimates of the rainwater available in any area of the country. For more accurate computations however, specific rainfall data for the area in question should be obtained. This specific data can be obtained from the Meteorology department or from rain gauge stations around the country. A list of some of the rain gauge stations is in Appendix 3E. The annual rainfall at a 90% probability level should be regarded as the "dependable rainfall" for purposes of rainwater harvesting for domestic use.

3.6.2 Runoff Coefficients

Rain falling on the ground produces surface runoff whose quantity depends on the runoff coefficient of the area. The runoff coefficient varies with topography, land use, vegetation cover, soil type and moisture content of the soil. In selecting run off coefficients the future characteristics of the water shed are considered. If land use varies within a water shed consider the segments individually and use a weighted coefficient value to determine the total runoff for the watershed. The run-off coefficients in Table 3-8 should be used to estimate the proportion of rainfall which can be harvested as surface runoff.

Table 3-8: Runoff Coefficients Applicable to Different Surfaces

Surface Type	Run-Off Coefficient
Roof tiles, corrugated sheets, bitumen, plastic sheets	0.8
Brick pavement	0.6
Compacted soil	0.5
Uncovered Surface, flat terrain	0.3
Uncovered surface, slope 0-5%	0.4
Uncovered surface, slope 5-10%	0.5
Uncovered surface, slope > 10%	>0.5

3.6.3 Determining Surface Runoff

3.6.3.1 Introduction

The Rational Method is widely used to determine surface runoff for small size catchment (Catchments of size < 25 km^2). The rational method is based on the assumption that a constant rainfall intensity is spread over an area and the effective rainfall falling on the most remote parts of the basin take a certain period of time known as the time of concentration (t_c) to arrive at the basin outlet. The relationship for peak runoff rates is given by:

$$\mathbf{Q}(T)=ADC(A)i(T,t_c)\sum_j C_j A_j \qquad (\mathbf{Bauwens, 2010})$$

Equation 3-2

Source: W. Bauwens 2010, Surface water Hydrology, Department of Hydrology and Hydraulic Engineering Faculty of Applied Sciences, Free University Brussels

Where Q (T) is peak flow rate with return period T, ADC(A) is areal rainfall distribution coefficient(-), $i(T,t_c)$ is rainfall intensity with return period T and duration equal to the concentration time t_c, C_j is runoff coefficient for the area j (-) and $\mathbf{A_j}$ is the surface of area j (m^2). The assumptions considered in the application of this formula are that rainfall-runoff process is a linear process, return period of the runoff is equal to the return period of the rainfall and dynamic behaviour of the flow and the storage are neglected. To satisfy these assumptions therefore, the considered area should be small (less than 25 km^2) and homogeneous with free outflow conditions with no backwater, no hydraulic structures, no storage reservoirs and no pumping stations.

The areal rainfall distribution coefficient ADC(A) is a correction factor used to account for decrease in total precipitation with respect to the areal extent. A return period is a time interval between rainfall events of certain intensity or size. Concentration time t_c is the time for a drop of water to flow from the remotest point in the watershed to the point of interest (Chow 1998). The dimensionless runoff coefficient is that part of rainfall that contributes to surface runoff. For design purposes to impervious areas when simplified design methods are employed, a value of 0.8 is used (Berlamont 2005).

3.6.3.2 Procedure for Using Rational Method

i) Determine the watershed area;
ii) Determine the time of concentration, considering the future characteristics of the water shed. The commonly used equation to calculate time of concentration is Kirpich's or other formula; and
iii) Calculate the critical rainfall intensity that causes the rainfall to operate at a steady state. The duration of the storm must be at least the time of concentration otherwise the maximum flow would not be reached.

3.6.3.3 Limitations of the Rational Formula

The rational formula makes the following assumptions:

i) The predicted peak discharge has the same probability of occurrence (return period) as the rainfall intensity (I); and
ii) The run off coefficient (C) is constant during the rain storm and the recession time is equal to the time of rise.

Alternative methods of calculating runoff for larger areas can be used such as the use of Intensity Distribution curves to estimate the runoff of a region.

3.6.4 Collection Tanks

3.6.4.1 Introduction

The required capacity of the harvested rainwater collection tank should be determined using the available rainfall pattern data for the area concerned. Traditional methods of rainwater harvesting are still used at household level in Uganda especially in rural areas where grass-thatched huts are still common. These include the use of clay pots, saucepans and other containers to trap rain without a system of conveyance or an attempt is made at creating one. Some improved technologies used for harvesting rainwater water in households are presented below.

3.6.4.2 Rainwater Jar (Ferro cement Jar)

This is a low cost technology for rural communities with distant and inadequate water sources. They are normally constructed with a capacity of 400-1500 liters and which is meant to serve a household of six days or less. In Uganda the main material used for their construction is Ferro cement.

3.6.4.3 Ferro Cement Tank

Ferro cement tank provides a better solution for RWH than the “jar” but comes with a higher initial cost. These tanks are constructed with a plain or reinforced concrete base, cylindrical walls of Ferro cement and a roof of Ferro cement or mild steel sheeting. These tanks can be constructed for capacities of as high as 4,000 liters. Figure 3-9 shows an example of a completed Ferro cement tank.

Ferro cement tanks require minimal maintenance and can last indefinitely if properly constructed. The tanks have the advantage that cracks in the structure are arrested quickly and are usually minimal resulting in a water tight structure, Ferro cement has a high tensile strength of 35 MPa (Namaan, 2000) and to some reasonable limits, and it behaves like a homogeneous elastic material.

3.6.4.4 Partially Below the Ground Tanks (PBGs)

This is another type of Ferro cement tank in where the part of the tank is cast below the ground level. Instead of a tap, a hand pump is provided at the outlet. It has the advantages that it requires little or no space above the ground, it is generally cheaper due to lower material requirements and the surrounding ground gives support hence lower wall thickness.

3.6.4.5 Corrugated Galvanized Iron (GI) Tanks

GI tanks have been used for several years all over the world and are still a popular choice for water harvesting. They range from small drums to very large tanks used in arid areas of the world. In Uganda the commonly used tanks are manufactured locally by skilled artisans and are in varying sizes of capacity. Tanks can be delivered to site and installed quickly by a skilled person. The main disadvantage of these tanks is that they start to corrode after a short period of use, about two years.

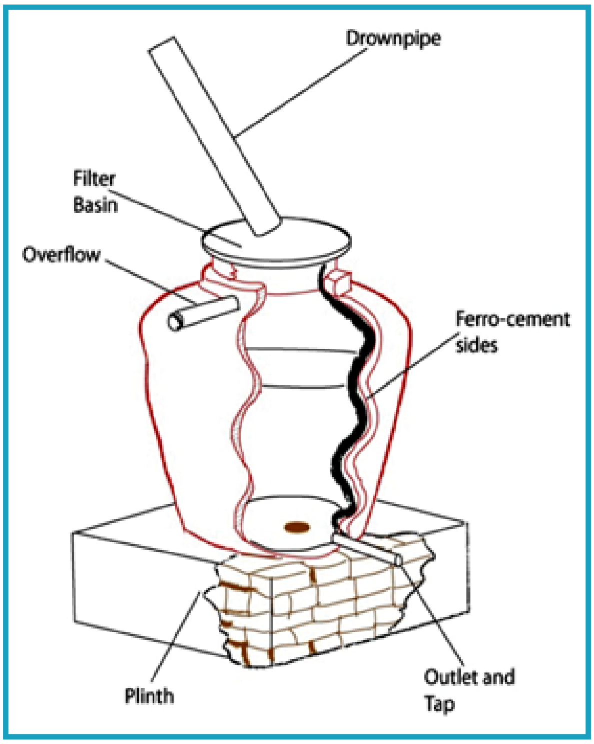


Figure 3-7: A Ferro cement Jar.
Source: DTU, 2000

3.6.4.6 Brick/Block Tanks

These tanks are constructed using locally obtained bricks or blocks and are therefore cheap to construct. The materials for the bricks and the bricks themselves may be prepared locally so that money is saved. Interlocking stabilized soil blocks may also be used however, a block making machine will be required to prepare the stabilized soil blocks. Because rectangular bricks are unsuitable for construction of a circular tank, pre-tensioned steel strips have to be incorporated in the structure so that it acts as a single unit carrying the tensile forces.

Block/brick tanks can also be expensive as the amount of mortar is determined by the brick thickness. If the bricks are thick, the demand for mortar will be higher thus increasing the cost of construction.

3.6.4.7 Sizing the Tanks

i) Supply Method - This is mainly used in areas that have uneven rainfall distribution or low rainfall. More care is taken in sizing the storage so that periods of scarcity are taken care of.

ii) Demand Side Approach - This method uses the consumption rates and occupancy to calculate the largest storage requirement. This method assumes sufficient rainfall and catchment area and is therefore suitable for areas with such conditions.

iii) Computer model - Several computer models exist that can be used to accurately estimate the tank size. One such model is SIMTANKA, developed by an Indian organization and is available on the internet for free (http://homepage.mac.com/vsvyas/science.html#simtanka). The program calculates the tank storage using a mathematical model of the actual system. It uses rainfall data to estimate the fluctuations in rainfall. A monthly record of rainfall data for at least 15 years is required for the model to work. If this data is unavailable, data similar to the area for which the system is being designed is used. It also calculates the reliability of the system based on the data provided.

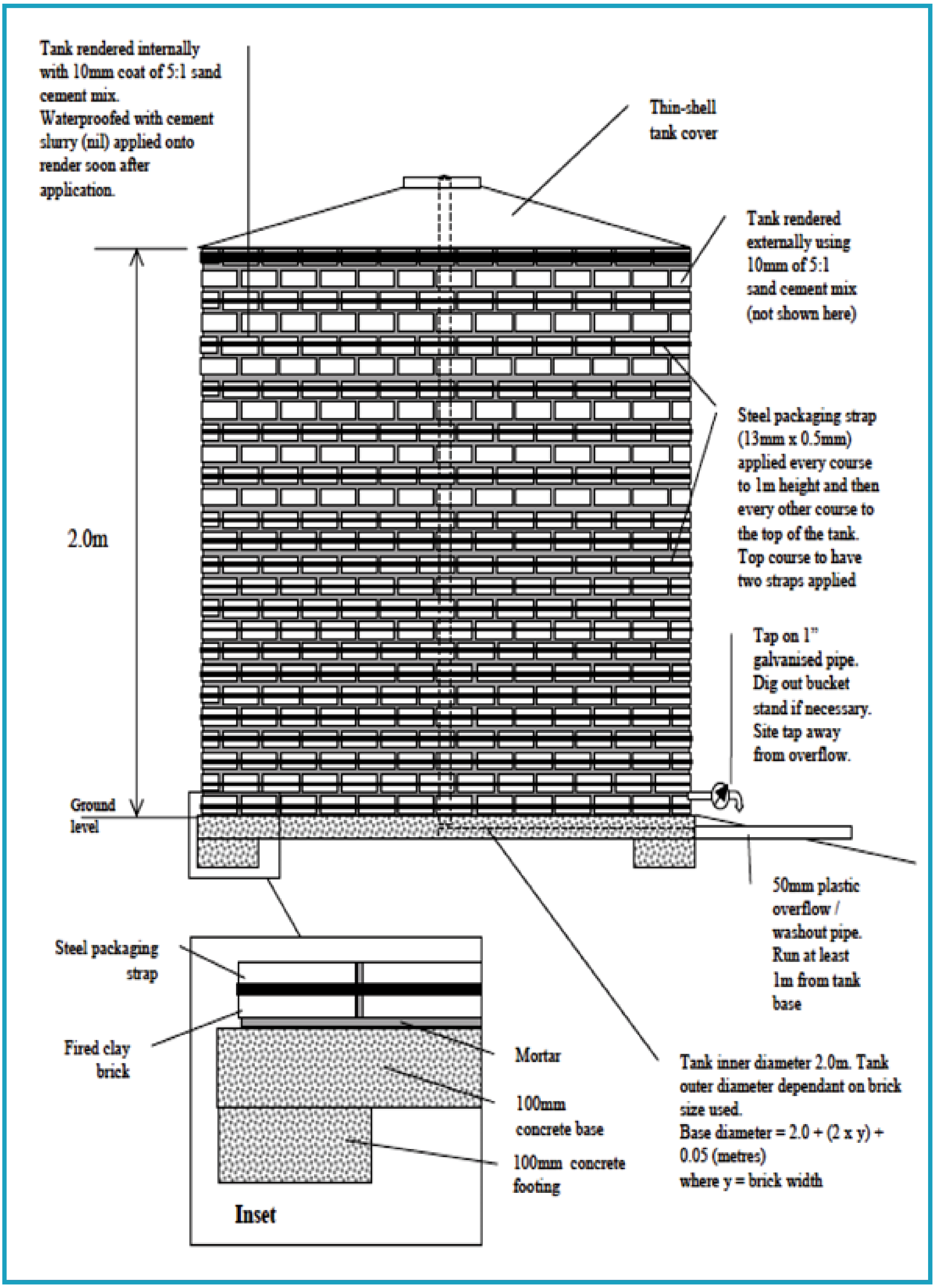


Figure 3 8: Main Components of a Single Skin Reinforced Brick Tank
Source: DTU, University of Warwick, 2000

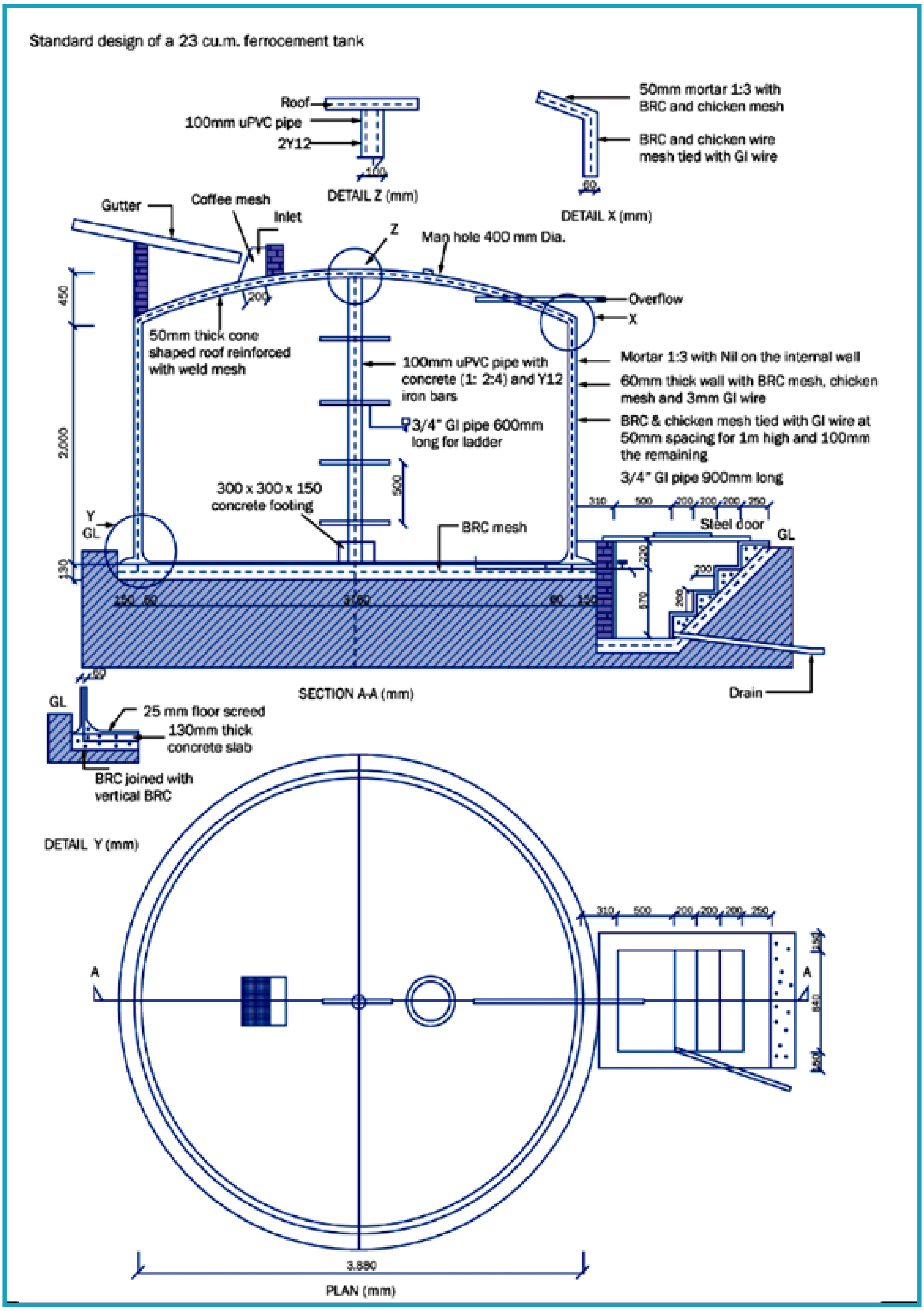


Figure 3 9: Design of a Ferrocement Tank
Source: Water Aid: Technical Brief, 2008

3.7 Roof Catchments

The preliminary estimate of the minimum roof area required to meet a given total water demand by rainwater harvesting can be made using a simple formula that uses the 90% probability annual rainfall.

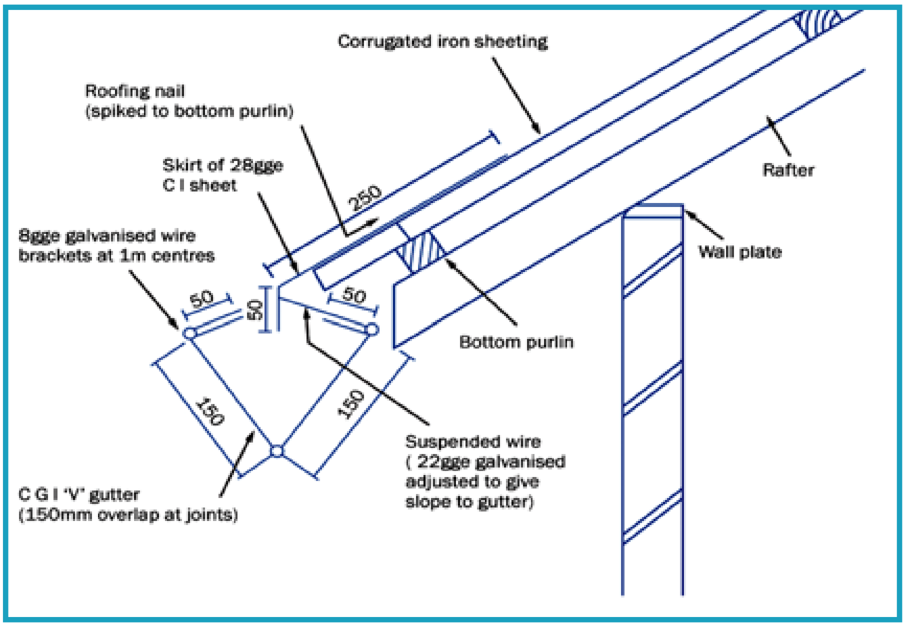


Figure 3 10: Section through a Typical Gutter

Source: Water Aid, Technology Notes, 2007

3.8 Ground Catchments

Tiles, corrugated iron sheets, asphalt, brick pavements and rubber or plastic sheets can be used to cover the ground to create rainwater harvesting catchment areas as shown in Figure 3-11.

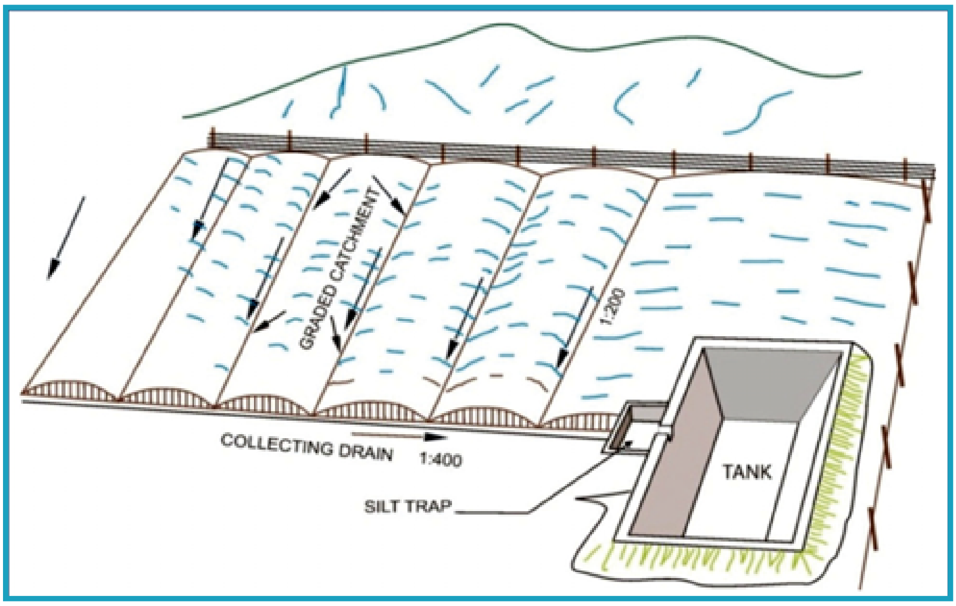


Figure 3-11: Ground Catchment.

3.9 Source Protection

3.9.1 Introduction

Source water protection refers to efforts to protect drinking water sources by minimizing the degree of contamination of a water source through application of appropriate management practices, the degree of treatment to achieve the desired water quality and safety requirements. Both surface and ground water sources require source protection plans.

Water sources are protected in order to:

i) Prevent threats to public health;
ii) Avoid extensive treatment and relocation costs; and
iii) Protect the only available water sources.

The following steps are recommended in implementing a source water protection plan:

i) Form a committee;
ii) Delineate the boundary – for surface water the area that contributes water to the surface water body must be delineated so that the potential contaminants are identified and managed. In delineating ground water sources the area that contributes water to the well must be protected so that the potential contaminants are identified and managed. The protected area is called the Well Head Protection Area (WHPA) and is divided into smaller zones;
iii) Assess the risks;
iv) Form a protection management plan; and
v) Monitor implementation of the plan.

3.9.2 Zone Protection Concept

The zone protection concept works to achieve the following levels of protection:

i) The zone directly adjacent to the site of the borehole and well to prevent rapid ingress of contaminants or damage to the wellhead;
ii) The Inner protection zone which is based on the accepted time required for pathogens to reduce to an acceptable level;
iii) The outer protection zone, which aims to achieve an acceptable level of dilution and attenuation of slowly degrading substances. Another consideration in delineation of this zone is the need to implement further interventions for persistent contaminants; and
iv) The larger outer zone covers the whole drinking water catchment area of an abstraction point. It is created to prevent long term degradation of water quality.

The process of zone protection ground water sources involves delineation of the boundaries. The considerations in zoning ground water sources include:

i) The distance of travel of the contaminant from the discharge point to the abstraction point;
ii) The travel time – the maximum time it takes to reach the abstraction point;
iii) The draw down - the extent to which pumping lowers the water table in an unconfined aquifer. This forms the zone of influence;
iv) Assimilative capacity – this is the degree to which the sub - surface may reduce the concentration of the contaminant; and
v) Flow boundaries – demarcation of recharge areas.

3.9.3 Protection Zones

3.9.3.1 Introduction

To minimize pollution and contamination of the sources exploited for water supply, it is recommended to establish and maintain protection zones around each source. A source should not be sited in areas where the ground water is likely to be contaminated. As a rule, a well site should be uphill and at least 50m from a point of contamination. The well site should not be subject to flooding at any other time. This will be of greatest concern where the source is in a low area or near a river that overflows its banks. However, The source maybe protected from flooding by building small dams or ditches. If not, another source should be considered (Water for the World, Technical note no. RWS.2.P.3). Two protection zone are recommended as discussed below.

3.9.3.2 Inner Protection Zone

The inner protection zone should cover an area of at least 50 m radius around the source. This area should be fenced off and it should have a lockable gate. The following restrictions and precautions should apply to this zone:

i) no pit latrines , septic tanks, etc., to be allowed within;
ii) no storage of fuels, oils etc., to be allowed within;
iii) access of humans and animals to be strictly controlled if at all permitted;
iv) all surface water runoff to be directed away;
v) dense vegetation to be encouraged and maintained; and
vi) Local communities to be educated about the importance of this zone.

3.9.3.3 Outer Protection Zone

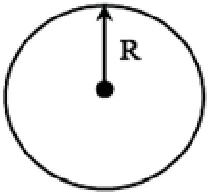
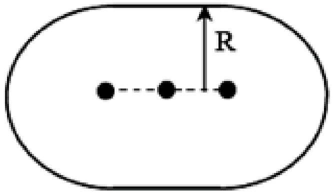
The outer protection zone should cover an area of about 300 m radius around the source. The following restrictions and precautions should apply to this zone:

i) no pit latrines or sewage tanks to be allowed within;
ii) no quarrying of rocks or soil to be allowed within;
iii) no petrol filling stations to be allowed within;
iv) no large public solid waste disposal facilities to be allowed within; and
v) Local communities to be educated about the importance of this zone.

3.9.3.4 Approximate equations for protection zones around pumping installation

Exact calculations of flow paths and travel times are usually very difficult. These require powerful computer models and extensive field investigations. Approximate equations can be obtained for simple situations and homogeneous groundwater systems. Figure 3 – 12 lists some equations for estimating the extent of protection zones around pumping installations, based on following assumptions:

i) all groundwater flows are considered radial around individual wells, or uniform aroundbatteries of wells;
ii) for travel times in the inner zone, no recharge is taken into consideration, all waterpumped comes from the groundwater layer, having a constant thickness and effectiveporosity;
iii) the size of the outer zone is obtained by considering an area such that the total recharge ratebecomes equal to the pumping, i.e. the total influence region.

Protection zone	One individual well or a cluster of wells 	Battery of wells 
1 or 2	$R = \sqrt{\frac{Qt}{\pi n D}}$	$R = \sqrt{\frac{Qt}{\pi n D} + \left(\frac{L}{\pi}\right)^2} - \frac{L}{\pi}$
3	$R = \sqrt{\frac{Q}{\pi q}}$	$R = \sqrt{\frac{Q}{\pi q} + \left(\frac{L}{\pi}\right)^2} - \frac{L}{\pi}$

R: maximum distance from one individual well or from the centre of a cluster of wells, or from the axis of a battery of wells [L]; Q: total pumping rate [L3/T]; t: travel time [T]; n: porosity [L3/L3]; D: thickness of the aquifer [L]; L: length of the battery; q: recharge rate [L/T].

Figure 3-12 Approximate Equations for Protection Zones Around Pumping Installation
Source: F. De Smedt 2007, Groundwater Hydrology, Department of Hydrology and Hydraulic Engineering Faculty of Applied Sciences, Free University Brussels

3.10 Water from the Mountains

Water from mountainous regions that are normally uninhabited by communities can be a useful source of water supply for communities that live close to or away from the mountains. Such water can be an option to supplement other sources or as a supply to areas without other sources. Rainfall on Mt. Elgon is on average 2,000 mm per annum. This is one of the highest rainfall areas in Uganda only next to the islands of Lake Victoria and the Rwenzori Mountains.

Table 3-9 Summary of Comparison of Methods of Water Sources

Method	Quality	Quantity	Accessibility	Reliability	Cost
Groundwater (Springs and Seeps)	Good quality; disinfection recommended after installation of spring protection.	Good with little variation for artesian flow springs; variable with seasonal fluctuations likely for gravity flow springs.	Storage necessary for community water supply; gravity flow delivery for easy community access	Good for artesian flow and gravity overflow; fair for gravity depression; little maintenance needed after installation.	Fairly low cost; with piped system costs will rise.
Ponds and Lakes	Fair to good in large ponds and lakes; poor to fair in smaller water bodies; treatment generally necessary.	Good available quantity; decrease during dry season.	Very accessible using intakes; pumping required for delivery system; storage required.	Fair to good; need for a good program of operation and maintenance for pumping and treatment systems.	Moderate to high because of need to pump and treat water.
Streams and Rivers	Good for mountain streams; poor for streams in lowland regions; treatment necessary.	Moderate: seasonal variation likely; some rivers and streams will dry up in dry season.	Generally good; need intake for both gravity flow and piped delivery.	Maintenance required for both type systems; much higher for pumped system; riverside well is a good reliable source.	Moderate to high depending on method; treatment and pumping expensive.
Rain Catchment	Fair to poor; disinfection necessary	Moderate and variable; supplies unavailable during dry season; storage necessary.	Good; cisterns located in yards of users; fair for ground catchments.	Must be rain; some maintenance required.	Low-moderate for roof catchments; high for ground catchments.

Sources: Water for the World, Methods of Developing Sources of Surface Water. Technical Note No. RWS.1.M

Chapter 4

WATER INTAKES

4.1 General

The purpose of a water supply intake is to extract and deliver water to the users. Therefore design of water intakes require a series of hydraulic design consideration in order to arrive at a desirable concept that can obtain and deliver the water economically with an acceptably low impact on the environment. Due to variability of site conditions, the challenge is in assessing water supply availability. The major factors that can affect the selection of a concept and design development for a water intake include:

i) water availability;
ii) Bathymetry;
iii) sediment transport;
iv) environmental regulations;
v) climatic conditions;
vi) Constructability;
vii) initial and maintenance requirements; and
viii) operation and maintenance.

4.2 Design Considerations

Of all the above factors, for design purposes, water availability is the most important. The extraction should be done in a sustainable manner without creating an environmentally and physically adverse effect on the water body. Therefore, detailed hydrologic studies including analysis of historic data must be performed. In areas where no historic data are available, rainfall data should be analyzed to determine rainfall frequency. Hydrologic modeling can be used to estimate the runoff.

Locating and selecting the specific type of intake requires adequate knowledge of the bathymetric condition of the river, estuary or sea bottom in the vicinity of the intake. Without this information, no specific intake concept can be selected. Making assumptions could lead to erroneous cost and schedule estimates for the project.

Other important factors to consider are any water withdrawal limitations as well as the feasibility of dredging and disposal of dredge spoil. In some situations, water may be physically available, however, because of water rights, water required for aquatic habitats or waste assimilation may not be legally available. In addition, dredging and disposal in areas where there are endangered species or contaminated soil, could be harmful to the environment. These factors and others could affect the selection of a desired intake site and may affect the feasibility of a project.

Construction, maintenance and access are also important factors to be considered in selecting the intake location. Availability of access road, potential for local and riverine flooding and access to the intake equipment all year round should be considered.

4.3 Design Concepts of Intake Structures

Experience in the design and operation of various water supply intakes indicates that no single design concept is suitable for all locations. Therefore, any intake design must be based on site specific information. This may not be possible at the planning phase of the project due to the absence of specific site data. Therefore, the design parameters must be develop from the limited data that may be available, and develop programs for the field data collection and analysis for use in detailed design.

Lack of site specific information generally occurs in many parts of Uganda where no historic data, studies or maps are available to help in the planning and design. The most practical approach for work under these conditions is to make a site visit and obtain aerial photographs. An important aspect of this effort is the identification of site conditions especially erosion and deposition. Aerial photos can best be utilized in assessing the presence of lake level changes and river meanders. It can therefore be concluded that:

Locating and designing a water supply intake requires careful consideration of hydrologic, environmental, geotechnical and economic factors.

i) Several types of intakes should be considered to meet various site conditions and operational requirements;
ii) Long term hydrologic data should be collected and analyzed to arrive at the most suitable and reliable concept; and
iii) Hydraulic analysis must be performed as an integral part of the intake design to provide flow free from objectionable conditions at the pumps.

4.4 Ownership of the Intake

To safeguard the ownership of the source, the following requirements should be met before a source is accepted for water supply:

i) For the large towns and urban centers the site for the source works should be acquired by the Water Authority (WA), with a sale agreement and land title in the names of the Water Authority. No development may start without the land, comprising all or most of the catchment, being legally owned by the WA;
ii) For the rural towns, a Community Letter of Agreement, signed by the landlord and the relevant community members, is the minimum acceptable evidence of ownership. The landlord must acknowledge that the land will perpetually belong to the government and that the operators of the water source will have unimpeded access to the water source at all times;
iii) The source location shall be subjected to a social and environment impact assessment (SEIA) and shall be approved for development of the water source through the SEIA process. All the resettlement action plan (RAP) actions, including compensation of the landlords and the resettlement of the project affected community must be completed before the works can commence;
iv) The site should not be located within a protected area such as a gazette forest reserve or Game Park since access to this source may not be guaranteed. However, if that is the best of possible sources, the WA shall obtain written permission from Uganda Wildlife Authority (UWA), National Forestry Authority (NFA) or the Wetlands Division, granting the WA access to the source at all times and giving perpetual ownership to the source area;
v) Where only part of the micro-catchment is taken for the water source, the occupants within the remainder of the catchment must commit themselves, in writing, in the Community Letter of Agreement, to abide by the sanitation and hygiene requirements to ensure that source is not polluted. Requirements may include not erecting pit latrines, septic tanks or rubbish pits etc.; not to use certain farming methods e.g. fertilizers and pesticides within the catchment and not to erect other structures or use that may interfere with the quality of the source; and
vi) The aims of the above requirements are to reduce to the minimum disputes regarding ownership of water source works, access thereto and to clearly define the roles and responsibilities of the landlord, the community and the Water Authority in regard to the water source.

4.5 Strategic Considerations

Strategic considerations are important in siting the river and lake intakes. Damage or destruction of a water treatment plant may take several years to recover from, if at all. Certain types of pollution may render the whole body of water permanently unsuitable for use.

For the large towns, it is important to consider the possibility of the security of the water source, especially in relation to:

i) Possible pollution either from accidents or from malicious attacks;
ii) Possible damage to the intake and other structures, either from accidents or from malicious attacks; and
iii) The detection, elimination or mitigation of the threats posed to the raw water, the intake and related infrastructure.

Consultations should be held with district or regional security officials and their considered advice must be taken into account in deciding the final location of the water intake.

4.6 River Intakes

4.6.1 Siting

A river intake should, whenever practicable, be sited as follows:

i) on a river having a forested catchment area;
ii) on a level which allows the water to be supplied by gravity;
iii) upstream of populated and farming areas;
iv) upstream of bridges, cattle watering, laundry washing and sewerage outlet points;
v) upstream of industrial sites or areas;
vi) at a location where the area immediately upstream of the intake is not easily accessible to people and animals. Otherwise, the intake should be fenced off;
vii) where the ground is rocky or firm and does not get flooded;on the outside of a river bend; and
viii) where the flow is adequate to cater for the "Ultimate" year water demand.

4.6.2 Protection of the Water Source

For water sources for both large towns and urban centres and the rural water supplies, the WA must aim to acquire the entire micro-catchment area so as to reduce pollution to the water source. The whole micro-catchment should preferably be fenced and should be guarded as much as is practicable. Storm water drains should be constructed along the boundary of the source works to reduce the inflows of storm water into the source area. Flooding can destroy the source works or bring in debris and pollution to the area.

Where the conventional treatment plant requires the setting up of staff housing at the water works, the housing must be minimal within the micro-catchment area. Only essential staff should live within the micro-catchment. Disposal of excreta, solid waste, chemicals, etc. must be done outside the micro-catchment. Where storage of wastes is done within the micro-catchment area, great care must be exercised to reduce pollution of the groundwater and raw water.

4.6.3 Structures

For some water supply schemes, a weir or dam may be required to be constructed across the river to impound water in a reservoir, so that the quantity and depth of water available for supply is sufficient even during seasons of low flow. For details on the design of weirs and dams, reference should be made to internationally recognize standard design handbooks.

The intake should be designed such that clogging is prevented, scouring avoided and the structure remains stable even under flood conditions. The intake draw-off should be perpendicular to the direction of flow of the river. The bottom of the intake should be positioned at least 1 m above the river bed, to prevent rolling stones from destroying the intake screen.

A special baffle may be needed to keep out debris and floating matter such as tree trunks and branches. To reduce the amount of silt and suspended material entering the intake, the water should flow into the intake at a velocity not exceeding 0.1 m/s. After the water has passed the intake screens, the velocity should rise to at least 0.5 m/s to prevent the settling out of suspended matter. This minimum velocity must be upheld in intake chambers, canals and intake pipes even during the initial phases of a scheme, when the water demand is still low.

There should be facilities for the closing off of the intake using stop logs or other similar devices. In relatively large rivers with variable water levels, a floating intake may be the viable alternative. Pumping is usually required at river water intakes. If the difference between high and low water levels in the river does not exceed 4 m, a suction pump placed on the river bank may be used. Otherwise, an arrangement such as that shown in Figure 4-1 is recommended, with a sump constructed in the river bank and infiltration drains laid under the river bed. The river water percolates down to the drains and flows by gravity in to the sump. In this process, the river water is filtered and the suspended material content is also reduced.

As the lowest water level in the sump is likely to be too deep for a suction pump placed above-ground, the water is usually abstracted using a submersible or shaft driven pump placed in the sump.

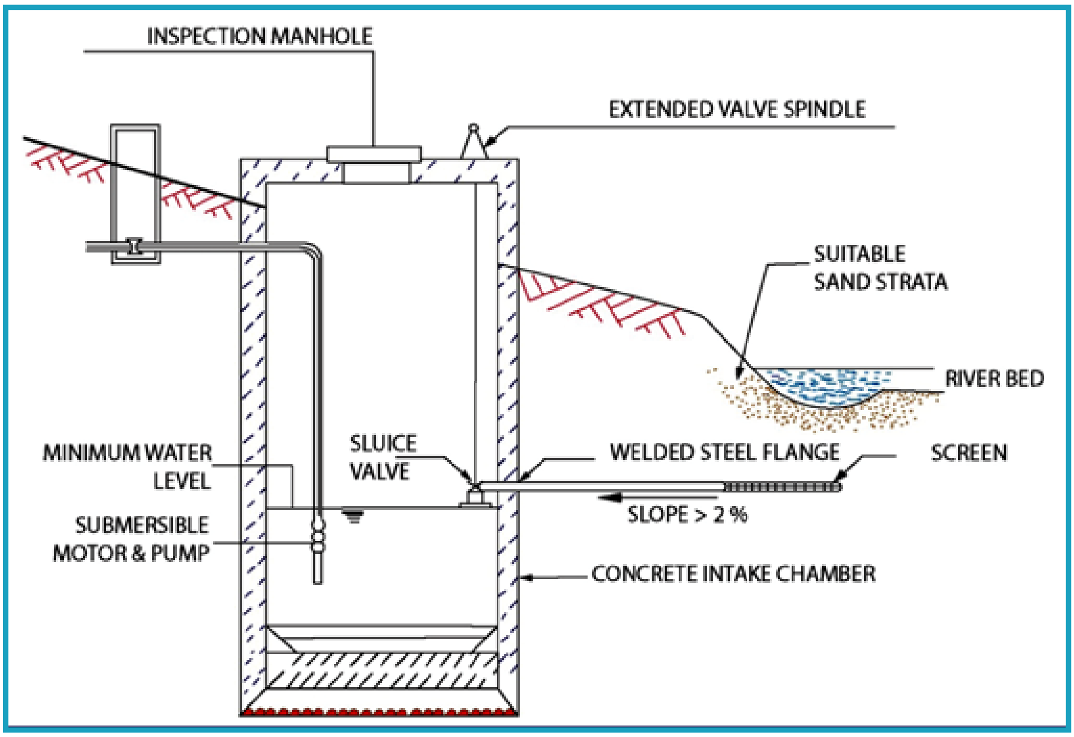


Figure 4-1: River Bank Intake.

4.6.4 Intake Screens

The intake should be equipped with a coarse screen and a fine screen, both of which should be removable. The coarse screen should have an open spacing of 30-50 mm between the bars. The fine screen should have an open spacing of 5-10 mm between the bars. The bars should be placed at an angle of 30° - 45° to the horizontal. It should be possible to clean the screens using a rake. Thus, the mesh type of screen is not recommended. The screens should be designed with a maximum water flow velocity in the openings between the bars of 0.7 m/s, otherwise soft, deformable matter will be forced through.

The head loss through the screen can be calculated using Kirschmer's formula[1]. It is good practice to design screens for a head loss of 0.5 - 1.0 m. The intake should be equipped with a platform, handrails and equipment to lift the screens.

4.7 Lake Intakes

4.7.1 Siting

The intake should be sited 3 - 5 m below the water surface where the oxygen content of the water is sufficiently high and the water relatively cool. However to avoid the entrance of silt, the intake should not be less than 1 m above the bottom of the lake. In lakes like Victoria with water is infested with schistosomiasis, the intake point should be sited some distance away from the shoreline.

The lake intake should be located as far as possible from shipping lanes, industries, or the delta or discharge point for a large river. River deltas can often get polluted, especially with suspended matter, pollution from factories or from human wastes originating upstream. Ships may carry hazardous materials and could run aground near the plant; they can also crash into the intake during storms.

Lake intakes should be located in secluded areas that are secure from short term pollution such as storm water. Where the possibility of long term pollution of the raw water exists, isolation of large bodies of water (reservoirs) using mobile gates should be considered. The water isolated behind the gates should be able to serve the requirements for the maximum duration of the possible pollution of the source, or until another source is found. The cost of such investment should be considered as a disincentive to locating the lake intake in areas that are prone to accidents or heavy pollution. Another disincentive is the cumulative cost of treatment that may be reduced significantly with a well thought out location for the treatment plant.

Examples of lake - intake structures include:

i) Variable depth intake structures; and
ii) Multi-level water intake structures.

4.7.2 Design Details

The underwater pipeline should be laid at an even slope with no peaks where air pockets can be formed. The pipeline should be adequately flexible and anchored to prevent buoyancy especially when the pipeline is empty. The cleaning of the intake screens should be considered in the design. Where feasible, a connection should be made from the discharge pipe to the underwater pipe to make the backwashing of the intake pipe possible. In lakes like Victoria where water levels fluctuate considerably, the level of the intake should similarly be adjustable. Water should flow towards and into the intake at a velocity not exceeding 0.1 m/s.

4.8 Boreholes

4.8.1 General

The design and construction of water supply boreholes in Uganda, has now become fairly standardized as follows:

i) a borehole is drilled with a final diameter of 150 - 300 mm by percussion or rotary/percussion methods;
ii) a casing is installed down to the un-weathered solid bedrock; and
iii) in unconsolidated sediments, a continuous slot, wedge or wire screen is installed at the aquifer together with a gravel pack.

[1]Kirschmer's formula is a general formula with several variations and simplifications. See FAO in Hydraulic Formulas Used in Designing Fish Farms for example.

A sufficient diameter should be provided to allow pump installation above and below the water aquifer. The intake of a borehole pump should be set at least 2 m above the bottom of the borehole.

In all production boreholes, sufficient space should be provided for a separate 20 mm diameter access pipe to accommodate water level monitoring equipment. This will also require an extra hole to be provided in the well head. Special precautions must be taken against the risk of pollutants entering the borehole through this extra hole.

4.8.2 Siting of Boreholes

The borehole siting methodology should be adjusted to the hydrogeological conditions and the local experience and should be done by an experienced Hydro-geologist. It should include the following steps:

i) Identification of fracture zone on aerial photographs, satellite images and maps;
ii) Identification of fracture zones in the field using resistivity profiling; and
iii) Electrical profiling.

Success rates using the Precise Location of Resistivity Anomaly (PLRA) method have been found higher than other methods; it is therefore the recommended method, especially for areas of low groundwater potential. The chosen sites should be subjected to an environment scoping or an environment impact assessment as required under the National Environment Act. Failed boreholes should be filled in with selected sand and gravel and topped up with grout or given other treatment to ensure that pollution does not enter into the aquifer through the borehole.

4.8.3 Borehole Types

4.8.3.1 General

There are two major types of boreholes commonly used in Uganda namely “open unscreened boreholes” and “screened boreholes”.

4.8.3.2 Open Unscreened Boreholes

Open, unscreened boreholes: in stable rock, with a casing placed in the over burden only. Open unscreened boreholes are normally constructed in stable formations as shown in Figure 4-2. In hard bedrock, the boreholes normally pass through overburden which needs to be cased to prevent it from collapsing into the borehole.

The casing must be installed at least 3 m into the firm bedrock and sealed by cement grouting. The boreholes should be drilled at least 6 m below the water bearing fractures, to allow sufficient space for a sedimentation sump.

4.8.3.3 Screened Boreholes

Screened boreholes are constructed in unconsolidated sediments, soft or highly weathered bedrock, supported by a casing and a screen. Screened boreholes with gravel packs are used in unconsolidated formations such as sands and sandstones. A continuous slot, wedge or wire screen is installed at the aquifer as a support for the natural formation, together with a gravel pack. A plain casing is extended upwards to ground level. A typical screened borehole is shown in Figure 4-3.

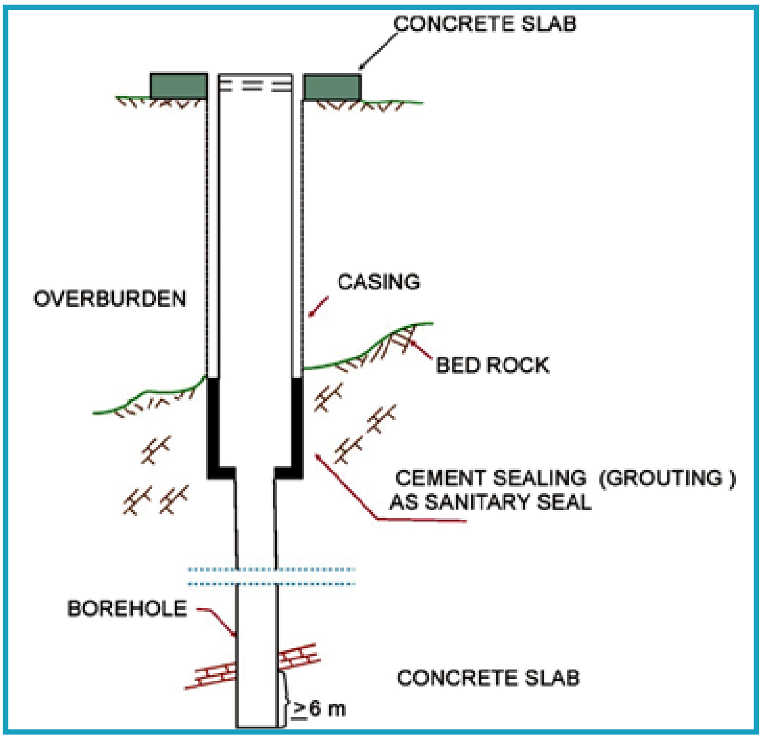


Figure 4-2: Open Unscreened Borehole.

A screened borehole consists of three main components, namely the casing, the screen and the sump. The sump should have a plain casing of the same diameter as the screen and extending at least 6 m below the screen.The gravel pack is normally required behind the screen to prevent the sediment particles from the natural formation from entering the well.

The following criteria should be used for the design of the gravel pack:

- the thickness of the gravel pack to be 0.10 m.
- Dp = 5.0 x D60

Where:

Dp = Diameter of gravel pack particles.

D60 = sieve diameter through which 60% of the natural formation material can pass.

Gravel packs must consist of well-sorted, well-rounded, smooth and clean siliceous material.

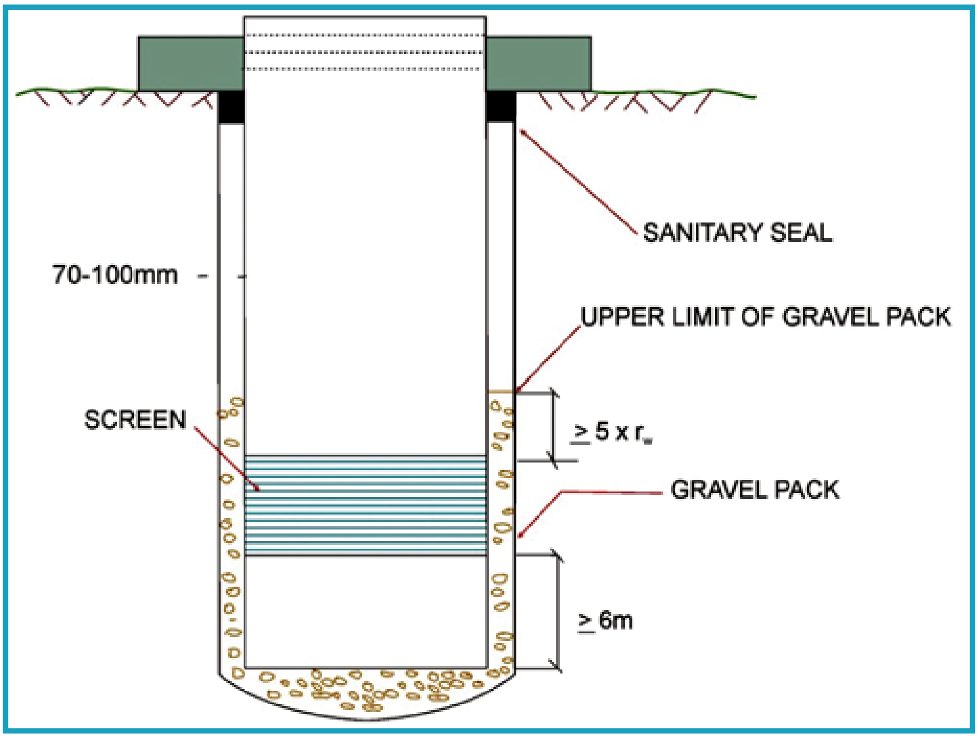


Figure 4-3: Screened Borehole.

4.8.4 Borehole Development

On completion of drilling and after the screen, casing and gravel pack have been installed the borehole must be fully developed. During this procedure, fine particles are washed out of the natural formation and the gravel pack brought into the well and then pumped out. As a result, a well-graded gravel pack will remain between the screen and the natural formation. A more detailed coverage of borehole capacity evaluation and development is coverered elsewhere in this manaul.

4.8.5 Borehole Development Techniques

Well development methods are based on establishing velocities of flow greater than those produced by the expected rate of pumping from the completed well. Ideally, this is combined with vigorous reversal of flow (surging) to prevent sand grains from bridging against each other (Schreurs, 1989). Movement in only one direction, as when pumping from the well, does not produce the proper development effect since sand grains can bridge voids around the screen. Agitation from pumping during normal pump use may cause these bridges to break down over time and sand to be pumped. This sand will act like sandpaper in the pump cylinder and will cause the cup leather to wear-out and the pump to fail within a few days or weeks.

The simplest but least effective development method is pumping a well at 2-3 times the designed discharge rate for a prolonged period. This does not really agitate the soil enough to create a real filter around the screen and it tends to develop only a short section of the length of screen (Anderson, 1993). However, it is useful because if the well can be pumped sand free at a high rate, it can be pumped sand free at a lower rate (Driscoll, 1986).

If the water level is within 3 –5m of ground surface, it is possible to use a mud pump as a suction pump to pump water from the well for 2 to 3 hours. However, do not pump the well continuously, start-stop cycle pumping is best for developing a well.

If this is not possible, install the bush pump and use a separate cylinder for the development process since particulate matter removed during development can cause an abnormally high rate of wear on the pump resulting in early pump failure. Using a larger pump cylinder than planned for the final installation will enhance the effectiveness of the well development.

Backwashingis alsoa relatively simple method of development which requires a water lifting device and a container in which water can be stored and then from which it will be allowed to flow easily back into the well. Water is pumped to the surface until the container is full.It is then rapidly dumped back into the well. Repeating this motion many times can provide some development of the surrounding water bearing formation. It is crucial that the water which is pumped to surface be allowed to sit until the suspended material has settled. The clear water should then be decanted into a second container and from there dumped back into the well. This will ensure that fine particulate is not inadvertently re-introduced into the well. If a gasket has not been attached to the top of the pump cylinder, it may be possible to combine overpumping with backwashing by collecting water from the overpumping process, allowing it to settle and then rapidly pouring the decanted water back into the well.

Surging is the most common method of well development. It involves forcefully moving water into and out of the well screen using one of the following techniques:

i) **Compressed Air**: Compressed air can be injected into the well to lift the water; As it reaches the top of the casing, the air supply is shut off, allowing the aerated water column to fall (process called “rawhiding”). The air supply should be periodically run without stopping to pump sediment from the well. This equipment is usually not available in remote areas and often only opens a small portion of the screen.

ii) **Bailer**: A bailer is like a length of pipe with a one-way valve in the bottom. The bailer is lowered into the well until it fills with water and sediment. it is then pulled to the surface and emptied. Water from the aquifer will then flow towards the well and bring in more drilling fluid. A bailors up-and-down motion causes a surging action which will develop the area around the screen. The heavier and wider the bailor is, the better it will function because it will have more force to push water through the screen (Brush, 1979).

iii) **Surge Block**: A surge block is a flat seal that closely fits the casing interior and is operated like a plunger beneath the water level. Because it seals closely to the casing, it has a very direct positive action on the movement in the well (Brush, 1979). Placing a surge block on the end of Water tubing equipped with a one way valve has the advantage of the down stroke being milder than the upstroke because some water passes up the tubing. This is advantageous because it ensures that fines are not driven further into the formation and it helps to remove sediment which is loosened by the surging action. This prevents the screen from becoming totally blocked with accumulated fines.

To effectively surge a well, apply an up and down motion, repeatedly raising and dropping the plunger 1 m. The plunger should drop rapidly on the downstroke in order that turbid water will be lifted out of the connecting tubing. While the plunger can be forced down on each stroke, adding weight just above the surge block will make it easier to work for a longer period of time.

Surging should start above the screen to reduce the possibility of “sand-locking” the surge block (Anderson, 1993). Initial surging should be with a long stroke and at a slow rate (20 to 25 strokes per minute). after surging above the screen, the hole should be cleaned and surging started at the lower end of the screen, gradually working upward until the entire screen has been developed (Anderson, 1993). When the amount of fine material drawn into the well begins to decrease, the process should be repeated, beginning at the bottom of the screen, but with a faster stroke (30 to 35 strokes per minute). The final surging should be as rapid as possible for as long as possible until the water coming out is of acceptable quality.

4.9 Spring Intakes

A spring chosen for water supply should be enclosed in a structure (for sanitary protection to prevent the contamination of the water) from which a pipe leads down conveying the water to the point of delivery.

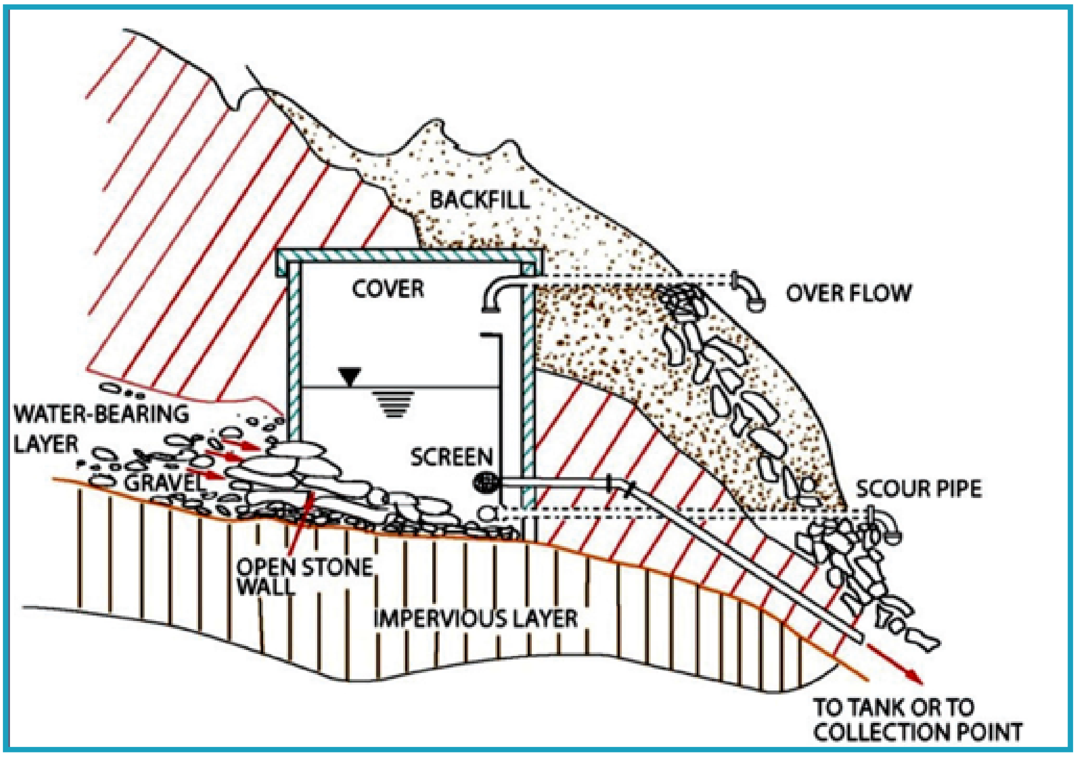


Figure 4-4A Typical Structure Used for Tapping Springs

Springs in granular ground formations can be tapped using infiltration drains consisting of pipes with open joints, placed in a gravel pack. To protect the spring, it is necessary to dig into the hillside so that a sufficient depth of the aquifer is tapped even when the groundwater table is low. The infiltration drains should be laid deep such that the saturated ground above them acts as a storage reservoir compensating for the fluctuations in the groundwater table. The water collected by the drains discharges into a storage chamber (“spring box”).

As can be seen above, the construction materials for springs include concrete masonry and geologically hardened abrasion resistant stones. This material is corrosive resistant and has minimal effect on water quality. The drain system and storage chamber should be so designed and constructed as to prevent pollution and contamination of the collected water. Thus, the top of the drain gravel pack should be at least 3 m below ground surface. The storage chamber should be fitted with a lockable manhole cover for access for cleaning and maintenance work. Air vents, overflow pipes and clean-out drains must have screened openings.The spring site should be fenced off and upstream, a cut-off drainage ditch is required to divert surface water runoff away from the storage chamber.

4.10 Roof Catchments

4.10.1 General

Details regarding the quantities of rainwater available for harvesting, the required roof areas and the capacities of storage tanks are contained in Chapter 3 – “Water Sources”.

4.10.2 Roofs

Rainwater can be collected from house roofs made of tiles, slates, corrugated galvanized iron or aluminium sheets. Thatched and asbestos cement roofs are not suitable because of the health hazards associated with them.

4.10.3 Roof Guttering

Roof guttering should slope evenly towards the downpipe to prevent the formation of pools where mosquitoes can breed. New houses should be carefully planned such that the length of guttering and pipes is as short as possible, and water can be tapped by gravity.

4.10.4 Foul Flush

Dust, dead leaves and bird droppings will accumulate on house roofs during the dry periods. This dirt will be washed off the roofs by the first new rains. To prevent pollution there should be an arrangement for preventing the first rainwater from being collected in the rain water storage tank. This is accomplished by having a vessel that collects the foul flush before water starts overflowing into the rain water storage tank as shown in Figure 4-5. Volume sizing of the foul flush can be done using the rational method (equation 4-1) as given below.

$$V = \frac{Q}{T} = CIA \quad \textbf{Equation 4-1}$$

Where V is the volume of the foul flush, Q is discharge, T is the duration, C is runoff coefficient (as given in table 3-8), I is rainfall intensity and A is roof area. Allow for collection of the first 5 - 10 minutes of rainfall for clensing the roof surfaces. A foul flush vessel of 100 - 200 liters should be adequate for an ordinary roof.

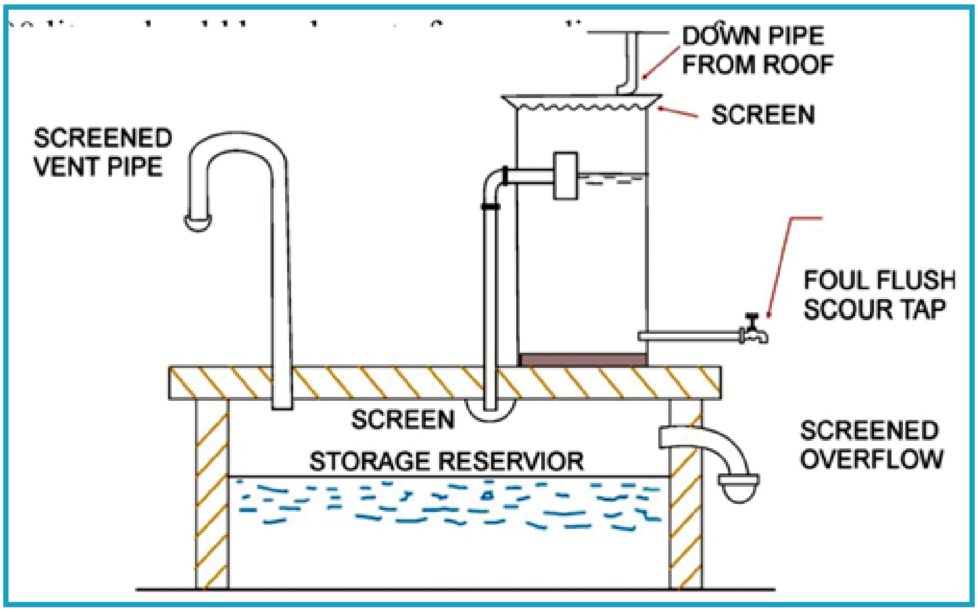


Figure 4-5: Foul Flush Arrangement for Roof Catchments.

4.10.5 Rainwater Storage Tank

4.10.5.1 General

The required capacity of the rainwater storage tank can be determined as discussed in Chapter 3 – "Water Sources". The inlet pipe should be equipped with a sieve or net material to trap foreign matter. The tank should be covered to reduce contamination and evaporation losses.

The outlet pipe should be placed 0.2 m above the floor of the tank. The tank should have a scour or be constructed in a manner which facilitates cleaning and removal of sediment. The tank should be well-raised to facilitate tapping. The tap area should be properly drained and it should also have a concrete apron to keep it dry and clean.

4.10.5.2 Plastic Storage Tanks

Plastic storage tanks have become the dominant type of tank; however, the major weakness is the durability of the tapping. If the tapping breaks off, the whole tank is rendered non-functional since it cannot retain water. Tanks should therefore be reinforced with gusset plates of sufficient dimension to ensure that the point of attachment is sufficiently strong; ensure that the gusset plates can be removed and replaced irrespective of the condition of the tapping hole.

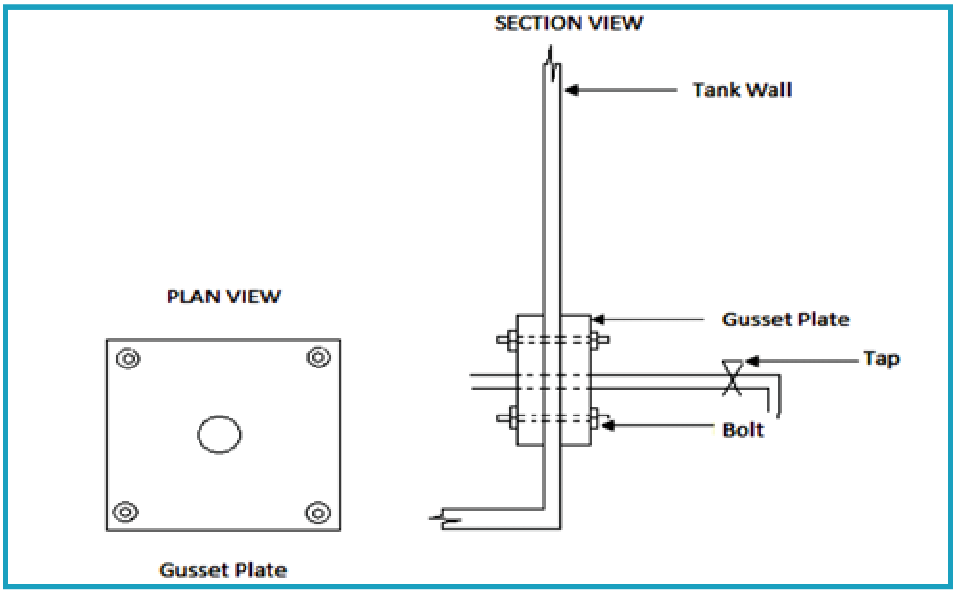


Figure 4-6: Gusset Plate Reinforcement of Tap Attachment to a Plastic Storage Tank.

4.10.5.3 Ferro Cement Water Tanks

Ferro cement water storage tanks are a low cost solution to storage of rainwater and are increasingly popular. Construction of Ferro cement tanks should be carefully planned, ensuring that the load and stresses do not exceed the strengths of the materials used. Resource materials include, for example, the UNHCR document found at Large Ferro cement Water Tanks (Design Parameters and Construction Details[2]). Owing to the low cost of the tanks and the fact that they can be constructed with mostly locally available materials such as chicken wire mesh, sand-cement mortar and with little technical knowledge, they should be encouraged for rural water supplies and at the household level.

[2] It is important to note that the design of Ferro cement water tanks is not well developed. The design and construction of these tanks should therefore be done by experienced artisans. Several smaller tanks should be chosen over few large tanks to reduce the risk of catastrophic failure which may lead to water shortages for long periods.

4.10.5.4 Corrugated Iron Water Tanks

Corrugated iron water storage tanks are made of locally available materials. However, owing to the low quality of the corrugated iron materials, such tanks should be used only at the household level. Most manufacture of these tanks is done by local artisans with little regard to the quality of the iron sheets and welding skills necessary, leading to rusting and perforations of the tanks.

Corrugated galvanized iron tanks should be laid on wooden supports placed on raised concrete platforms, to ensure that the outer bottom surface is kept dry and thereby reducing the chances of corrosion occurring.

4.10.5.5 Brick and Concrete Tanks

Brick or concrete tanks may be constructed underground, at ground level or elevated. Because of abundance of skills for the construction of brick or concrete tanks, they are recommended for both urban and rural water supplies. Quality control is of paramount importance to ensure that the joints are made leakage resistant and waterproofing is done for the walls and base. The choice of tanks to be used, especially for rural water supplies depends on the cost of the materials available, construction skills and durability of the final product.

4.10.6 Design

4.10.6.1 Introduction

For purposes of designing rainwater harvesting structures, 90% of the annual rainfall for the region can be considered as the dependable rainfall for domestic use. Specific rainfall data for particular locations should be obtained for individual cases. The following run-off coefficients should be used for calculating the fraction of rainfall that can be harvested using the rational method for pre design or IDF curves for detailed design.

4.10.6.2 Roof Catchment Sizing

The required roof catchment area (A) in $m^2$to meet total water demand (D) in litres/day given dependable rainfall R in mm can be calculated using the formula below

$$\mathbf{A = \frac{450D}{R}} \qquad \textbf{Equation 4-2}$$

4.10.6.3 Selection of Tank Size

The tank size will depend on majorly the total water demand (D) in litres/day and the rainfall pattern of individual areas. The formula below can be used to estimate the tank size

$$\mathbf{C = 0.03DT^2} \qquad \textbf{Equation 4-3}$$

Where T is the longest dry spell in months per average year and C is tank capacity in m^3. The definition of a dry spell may be taken as less than 50mm of rainfall per month.

To entirely depend on rain water harvesting, a large catchment area and storage is required especially where draught periods are long. The computation of the total water supply requirement in litres per month can be calculated as

$$\mathbf{D = 30NC} \qquad \textbf{Equation 4-4}$$

Where N is number of people to be supplied abd C is the per capita daily consumption (l/p/d). The common methods of determining tank sizes are further discussed below.

4.10.6.4 Balance Method

This method balances the yield, supply of water with the user or demand at the end of each month and calculates the storage left in the tank. This method can be used to determine the minimum tank size to satisfy water use for a family. An illustration is given here below for a family of 6 in Malera village, Bukedea district. Considering that annual rainfall is 567mm, roof Area $72m^2$, a family of 6 persons and a water use of 14 litres/person/day.

Table 4-1: Balance Method

Month	Rainfall R	Supply S (RxA)	Demand (D)	S-D	Cumulative (S-D)	S(I) (RxAxD)	S_m
	(mm)	(litres)	(litres)				
January	22	1425.6	2604	-1178.4	-1178.4	2008.8	0
February	365	23652	2352	21300	20121.6	1814.4	17
March	87	5637.6	2604	3033.6	23155.2	2008.8	17000
April	0	0	2520	-2520	20635.2	1944	14396
May	35	2268	2604	-336	20299.2	2008.8	14319
June	58	3758.4	2520	1238.4	21537.6	1944	15485
July	0	0	2604	-2604	18933.6	2008.8	12881
August	0	0	2604	-2604	16329.6	2008.8	10277
September	0	0	2520	-2520	13809.6	1944	7577
October	0	0	2604	-2604	11205.6	2008.8	5153
November	0	0	2520	-2520	8685.6	1944	2833
December	0	0	2604	-2604	6081.6	2008.8	113

The balance method formula for tank sizing is given by

$$S_m = S_i + I - D \qquad \textbf{Equation 4-5}$$

Where S_i is the stored water at the end of the previous month and is assumed never to be less than zero, I is the product of monthly rainfall, roof area and loss factor.

4.10.6.5 Cumulative Supply and Demand

In this method, the supply, demand and the monthly cumulative values are calculated and the maximum difference determined. This difference is the optimum tank size as illustrated below.

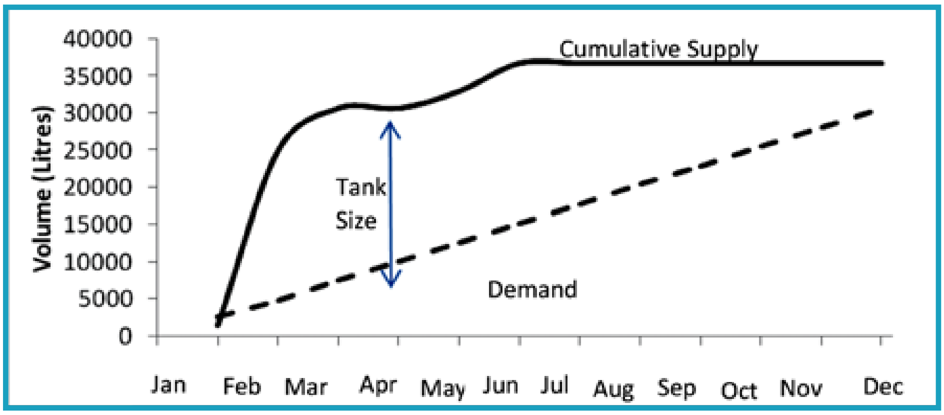


Figure 4-7: Cumulative supply and demand

The tank size is the optimum to collect the rainfall from a given roof area and satisfy water use by the family. For comparison when using similar input data as for the balance method, the optimum tank size is 23,000 Liters as seen in column cumulative (S-D) in Table 4.2.

4.10.6.6 Dry Season Storage Method

This method requires identification of longest period during the year without rain like July – December totaling to 120 days in the case above. Then the daily water use for the 6 persons at a rate of 14 litres/day is calculated as

$$\text{Tank size} = \text{No. of dry days x daily water use}$$

$$120 \text{ x } 14 \text{ x } 6 = 10080 \text{ litres } (10\text{m}^3)$$

If the annual yield is less than the dry season storage tank size, then the tank will have to be reduced to the value of the annual rainfall yield. The computation of tank size given annual rainfall of 567mm on a 15m^2 roof area and a collection efficiency of 90% is then calculated as:

$$0.567 \text{ x}15 \text{ x}0.9 = 7.6\text{m}^3$$

The tank size will have to be adjusted to 8m^3

4.10.6.7 Collecting and Storing all the Rainfall

In this case, all the rain is collected and stored only making it available when there is shortage. This implies that the tank size is equal to the total supply of the year. For a given annual rainfall of 567mm, roof area of 72m^2 and collection efficiency of 90%.

Tank size=annual rainfall x roof area x collection efficiency

$$0.567 \text{ x } 72 \text{ x } 0.9 = 36\text{m}^3$$

4.10.7 Conclusion

In general, the balance method allows for a minimum tank size to be determined that would satisfy daily demand especially in cases of limited resources to build the tank. It is therefore suitable for rural communities where rain water is considered as the only source of clean and safe water. The cumulative supply/demand approach allows for the ideal tank size to be estimated where funds are not limiting. Lastly, the dry days method is quick and easy but does not reflect rainfall patterns accurately.

Regardless of the approach, it is important to adhere to the following key points for the management of a successful rainwater catchment system.

i) Have gutters on the maximum roof area
ii) Maintain the gutters to collect the maximum amount of rainfall
iii) Use water economically especially towards the end of a dry season
iv) Clean gutters and tanks regularly especially after a dry spell to ensure that the water collected id of good quality. If funds allow, a foul flush system needs to be installed or use self-cleansing gutters

4.11 Ground Catchments

4.11.1 Introduction

For ground catchments, the ground surface should be prepared in such a way as to ensure relatively rapid flow of water to the point of collection and storage, in order to reduce evaporation and infiltration losses. The proportion of rainfall that can be harvested using ground catchments, ranges from about 30% for pervious, flat ground to over 90% for sloping ground covered with impervious materials. The speed of storm water entering the ground catchment and the surface determines the scour and therefore the silt load that enters the ground catchment. Care should be exercised to ensure that the approaches to the tank are planted with appropriate grasses or lined so that there is a reduced silt load at the tank. Details regarding the quantities of rainwater available for harvesting and the required ground catchment areas are presented in Chapter 3 – "Water Sources".

4.11.2 Dug Wells

4.11.2.1 Siting of Dug Wells

Dug wells are generally shallower than boreholes and will therefore generally be sited in areas with high water tables. The location should be carefully selected to ensure good potential for most of the year. The location selected should not be prone to flooding and pollution, especially by storm water from built up areas within the upper reaches of the micro-catchment. Cut off drains should be constructed if this is feared. No pit latrine or septic tank should be constructed upstream of the dug well.

To ensure long term good water quality and quantity, it is essential to tackle the pollution problem through sensitization of the community. The community should protect the catchment and avoid polluting it.

4.11.2.2 Diameter and Depth

The diameter of a dug well should be at least 1.2 m, to allow two people to work together during the digging exercise. A slightly smaller diameter may be used if the digging is to be done by one person only. For a large community (of 200 people or more), a well diameter of 2-3 m is recommended.

Increasing the size of the well further may not be cost-effective, as the additional water yield so obtained may not be much. The well should be dug at least 3 m below the expected lowest water level.

4.11.2.3 Lining

Most dug wells need an inner lining constructed of materials such as brick, stone, in-situ concrete rings or pre-cast concrete rings. Generally, the easiest and safest method of sinking a dug well is to excavate from the inside of precast concrete rings. In very loose soil, other methods such as hand-drilling (augering) should be employed. In consolidated ground, including rock formations, the well may stand unlined but the upper section should always have a lining.

The section of the well penetrating the aquifer requires a lining with openings or perforations to allow groundwater to enter. However, in fine sand aquifers, the lining should not have openings or perforations, so that water enters only through the bottom of the well. The bottom of the well should be covered with graded gravel consisting of a 150 mm thick layer of grain sizes 1 - 2 mm overlain by a 150 mm layer of grain sizes 4-8 mm and this should be topped by a 150 mm thick layer of grain sizes 20 - 30 mm, to form a proper filter.

4.11.2.4 Protection

The upper part of the lining should be water-tight, to a depth of several meters below the lowest drawdown water level in the well. The annular space between the well walls and the lining should be sealed with puddle clay or cement grout, from the ground surface to the top of the aquifer or to at least 2 m below the ground surface. The top of the lining should be extended to about 0.5 m above the ground surface level, to form a wall around the well.

A concrete apron should be constructed on the ground surface, extending about 2 m all around the well, and sloping outwards towards a drainage ditch with a suitable outfall. The well top should be sealed with a water-tight slab. A manhole that can be tightly and securely locked should be provided for the inspection and disinfection of the water in the well.

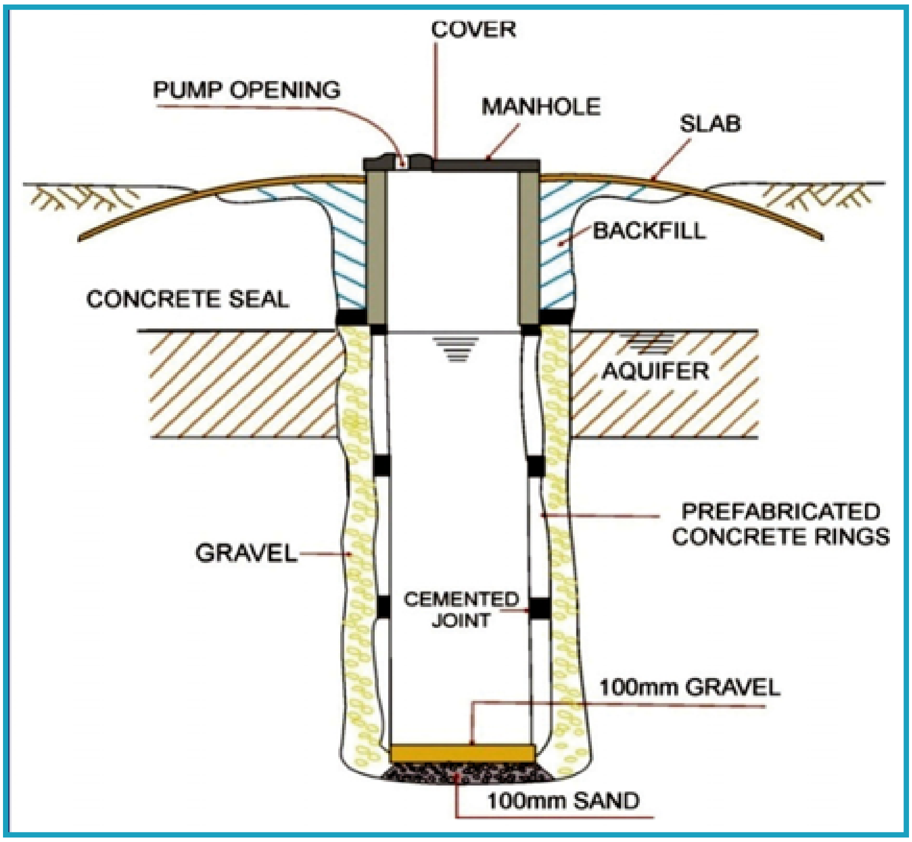


Figure 4-8: Construction Details of a Typical Dug Well

4.11.3 Infiltration Galleries

4.11.3.1 General

Infiltration galleries are horizontal ditches or drains for groundwater withdrawal, and they can be used in sand rivers, spring intakes and other types of groundwater abstraction works. A typical infiltration gallery is illustrated in Figure 4-9.

4.11.3.2 Length

For an infiltration gallery of the drain type placed under the bed of a river, lake or channel to abstract a given discharge of water for supply (see Figure 4-9), the required gallery length can be determined using the following simplified formula:

$$\mathbf{L = \frac{Qd}{KHb}} \qquad \textbf{Equation 4-6}$$

Where:

L = Length of the required gallery, m
Q = Desired discharge, m^3/s
d = Vertical distance between the riverbed and the center of screen, m
K = Permeability of the gravel backfill, m/s
H = Head of water acting on the center of the pipe, m
B = Average width of the trench backfill with gravel.

The infiltration gallery should be placed as deep as is physically and economically feasible beneath the bed of the river, lake or channel.

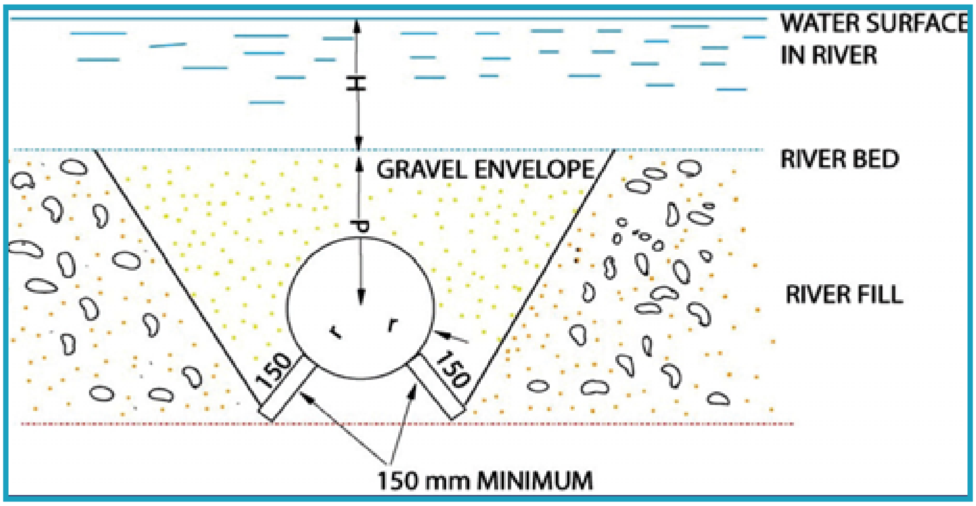


Figure 4-9: Infiltration Gallery under River Bed.

4.11.3.3 Drain Pipes

Drain pipes have pores, perforations or open joints to allow groundwater to enter. For the drain pipes to be self-cleansing, they should be designed for a water flow velocity of 0.5 - 1.0 m/s. The average entrance velocity of the water through the holes or slots in the pipe should not exceed 0.03 m/s.

The diameter of the holes or the width of the slots in the drain pipe should normally be 3 - 4 times the D60 of the gravel pack around the pipe. The D60 is the sieve diameter through which 60% of the gravel pack material can pass.

4.11.4 Sand River Intakes

Water can be extracted from sand rivers using well points, sunk into the river bed. Well-point systems are groups of closely spaced wells, usually connected to a common header pipe or manifold and pumped by suction lift. Commonly, small size well points, 50 mm in diameter which riser pipes are used as shown in Figure 4-10. In this case, the well point is the well screen while the riser pipe is the well casing.

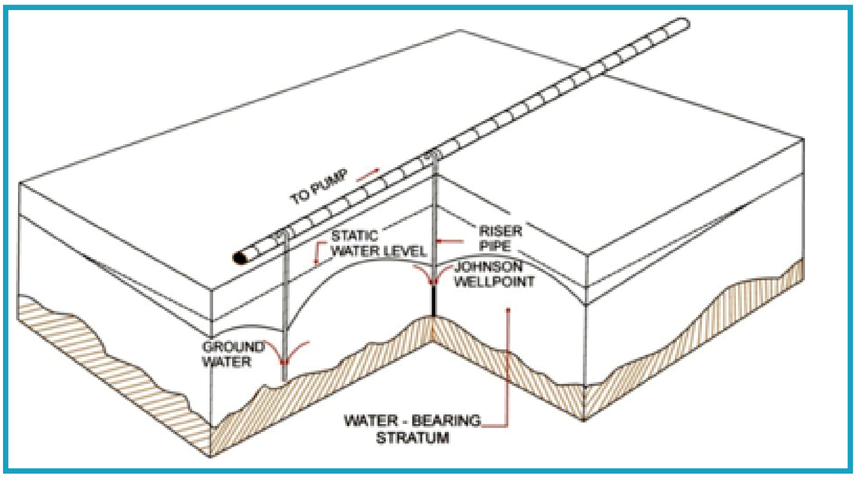


Figure 4-10: Well Point System

Chapter 5

WATER QUALITY

5.1 General

The quality of water supplied is an important consideration for any water supply development. The raw water quality must be determined in order to establish whether the water can be supplied in its raw form or whether it needs treatment. The level of treatment depends on the intended use of the water. The assessment of raw water quality is particularly important because it is the result of the water quality analysis that guides the decision on the treatment method required to treat the water to the set standards for its intended use. Water supplied for domestic use must meet the national water quality standards for drinking water before transmission and distribution to consumers. For Uganda the applicable standard is US 201: 2008 Uganda Standard for Drinking (Potable) water Second Edition published by the Uganda National Bureau of Standards (UNBS) in 2008. For water quality parameters where national standards do not exist, or outdated, WHO standards should be used.

A satisfactory water quality result does not make the water source perpetually good for consumption. Routine water quality monitoring is important as it informs the water operator of the possibility of pollution of the water source or the water produced.

The points of water quality considerations in the water supply include:

i) The source;
ii) The treatment; and
iii) The distribution.

For the purpose of water quality monitoring, sample taps are required at the following:

i) As close to the source as practical prior to any treatment; and
ii) Before the entry into a distribution system but after the treatment.

The minimum initial water quality testing may be dependent on a number of factors such as: location of the water supply system, the water source type i.e. surface water or groundwater, susceptibility of the water source to contamination process, water supply type, etc.

5.2 Guidelines and Regulations

It is common practice to assess potable water quality in relation to specific guidelines or standards. The formulation of such guidelines requires critical assessment of the properties of the various constituents of water.

The constituents are normally divided into five groups as follows:

i) Bacteriological (microbiological) parameters;
ii) Chemical parameters directly related to health;
iii) Chemical parameters indirectly related to health;
iv) Physical and chemical parameters related to aesthetic and technical effects; and
v) Physical and chemical parameters affecting building and pipe materials.

The World Health Organization (WHO) published the "Guidelines for International Standards of Drinking Water" in 1984 (The guidelines were revised most recently in 2011). The guidelines are intended as a basis for establishing local regulations after taking into consideration climatic, social, cultural and economic and other local factors. The Ministry of Water and Environment together with the UNBS have developed a National standard for drinking water which has to be complied with for all supplies for domestic water use.

The Uganda Standard US 201: 2008 specifies two classes of water:

- **Class I**: Potable water available from conventional treatment processes such as chlorination, sedimentation, filtration and ozonation. The quality specifications for this water are comparable to current international standards for water quality. This water is normally supplied through piped systems to urban areas and small towns.
- **Class II**: Potable untreated water available for consumers through point water sources such as boreholes, springs, shallow wells, harvested rain water and gravity flow schemes. This class specifies a water quality range that may pose a risk to consumers depending on the concentration of the contaminant within the specified range and the possibility of monitoring its quality. Regular monitoring of quality of this class is essential.

In general, water quality guidelines should always be applied with common sense, particularly for small community and rural water supply schemes where the choices for sources and the opportunities for treatment are limited. For small supplies which are frequently supplied from individual wells, boreholes or springs, the water quality criteria given in the guidelines, may have to be relaxed. But in all cases, everything possible should be done to limit the hazards of water contamination, using relatively simple measures like lining and covering the water sources.

5.3 Basic Requirements

According to the WHO guidelines, the basic requirements for drinking water are that it should be:

i) Free from pathogenic (disease causing) organisms;
ii) Free from compounds that have an adverse and acute or long term effect on human health;
iii) Fairly clear (i.e. low turbidity and little colour);
iv) Fresh (not saline or salty);
v) Free from compounds that cause offensive taste or odour;
vi) Incapable of causing corrosion or encrustation of the water supply system; and
vii) Incapable of staining clothes washed in it.

5.4 General Water Quality Issues

5.4.1 Point Source Pollution

One of the forms of point-source pollution results from the poor sewerage system coverage and lack of access to sanitation facilities in the country. This results in an increased incidence of pathogenic contamination which leads to outbreaks of water borne diseases such as Cholera and Dysentery. Another issue of importance is the increased level of nutrients and organic matter from urban and industrial point sources. This causes an increase in the level of eutrophication and oxygen deficiencies. Measurements from major towns served by NWSC have shown that the established regulations levels of BOD discharge are usually exceeded (WQMS, 2006). The low oxygen levels harm aquatic life and some of the blue – green algae is harmful to public health due to its toxicity.

Industrial discharges in the major towns in Uganda are not closely monitored and these result in dumping of poorly treated waste into rivers. Major potential polluters are breweries, textile industries, food and sugar processing, metal industries and leather tanning industries. The activities of these industries and other semi – cottage industries could pollute the water sources in the vicinity if they lack proper waste treatment systems.

Effluent and runoff from mining activities in some areas like Kilembe is discharged with high levels of heavy metal content. This affects the water quality from surrounding sources. The absence of river and effluent monitoring data makes it impossible to observe the effects on the water quality yet this water at some points may be fatal to both humans and bio – diversity. Other activities that may cause water quality issues are petroleum exploration such as in the Albertine region.

5.4.2 Non – Point Source Pollution

Waste water runoff from rural and urban areas may infiltrate into water sources and major water bodies and pose a risk of contamination. The water quality issue will be the risk of epidemics due to water borne diseases. The unregulated use of agricultural chemicals which leach into the soils and enter water sources presents another water quality issue of concern. The compounds from these fertilizers (Nitrates and Phosphates) aggravate eutrophication which further endangers aquatic life and public health moreover data on water quality around these sites is not readily available.

Another factor of importance is atmospheric deposition. Load estimates under Lake Victoria Environment Management Project (LVEMP), show that atmospheric deposition is a major contributor of nutrient pollution load (Nitrogen, 67% and Phosphorus, 81% of total load) into Lake Victoria (WQMS, 2006)

5.4.3 The Relation Between Contamination and Water Quality

Water quality is affected by the degree of contamination from the source to its point of consumption. Contaminants may be any physical, chemical, radiological or biological substance or matter in water. The water quality is determined by analysis of the mentioned parameters in the water in comparison to the allowable maximum contaminant level. The maximum contaminant level is the permissible level of contaminant delivered to the free flowing outlet of the ultimate user of the public water system except the turbidity which is measured at the point of entry into the distribution system.

Sources of water contamination include: - erosion of natural rock and soil formations, landfills, sewage, waste treatment facilities, discharge from industrial processes, farmland and homes and yards.

The following are the effects of contaminants on water quality:

i) Aesthetic – unappealing taste or odour and staining;
ii) Cosmetic – unappealing appearance;
iii) Acute health effects – these occur within a short period of consumption of the contaminant say days or hours; and
iv) Chronic health effects - these take place after consuming the contaminant for several years.

Water contaminants to be noted in water quality consideration include:

i) **Microorganisms:** not all microorganisms in water are harmful; however those that do will most likely cause acute health effects. They include protozoan parasites, algae, bacteria and viruses;
ii) **Radionuclides:** some minerals are radioactive and may emit alpha or beta particles and photons. Contamination of water by radionuclides results from erosion of natural or manmade deposits such as uranium. Consumption of contaminated water results in chronic health effects such as cancer and kidney problems;
iii) **Inorganics:** inorganic contaminants are mineral in origin and may occur from erosion of naturally occurring deposits, sewage, corrosion of pipes and plumbing and industrial wastes; and
iv) **Organics:** these are chemical compounds from carbon molecules. They may be natural or man – made such as synthetics. Common organics which may have potential health effects are herbicides and insecticides, runoff from landfills and industrial waste.

Other contaminants are disinfectants such as chlorine and chloramines and disinfectant by-products like bromate and chlorite.

5.4.4 Water Related Hygiene Problems

Water related hygiene problems are varied but can be classified by their transmission routes (White, Bradley and White, 1972).

i) Water borne - in which the disease causing pathogen is ingested in drinking water (feaco – oral). They include diarrhoea, dysentery, typhoid and cholera;
ii) Water washed – are favoured by inadequate hygiene conditions and practices and can be controlled by improvements in hygiene. They normally manifest as skin and eye infections. Scabies and Trachoma are some of the common water washed diseases;
iii) Water based – these are diseases transmitted by an aquatic invertebrate host. Examples are guinea worm and Schistosomiasis; and
iv) Water – related insect vector routes – they involve an insect vector that breeds in or near water. Some of the diseases are dengue, malaria and Trypanosomiasis.

Water borne diseases and water based diseases can be prevented by improvements in the water quality and availability. Improvements in water supply may reduce incidences of water – based disease while water related vector disease depends on the lifecycle of the vector. Other reductions in the incidences of water related hygiene problems can be effected by hand washing with soap. Figure 5-1 shows the relation between different sanitation interventions and morbidity from selected diseases.

Table 5-1 Potential Relation Between Water and Sanitation Interventions and Morbidity from Selected Diseases

Potential Relation between Water and Sanitation Interventions and Morbidity from Selected Diseases and Intervention				
	Improved Drinking Water	Water For Domestic Hygiene	Water For Personal Hygiene	Human Excreta Disposal
Ascariasis	+	++	-	++
Diarrhoeal Diseases	+	++	++	++
Dracunculiasis (Guinea Worm Disease)	++	-	-	-
Hookworm Infections	-	-	-	-
Schistosomiasis	-	++	++	++
Trachoma	-	+	++	-

Source: Potential Relation between Water and Sanitation Interventions and Morbidity from Selected Diseases, UNICEF Report 1993

Notes:
++++ = increasing positive effect
o = no effect
- =negative effect

5.5 Bacteriological Quality

5.5.1 General

Bacteriological quality is very essential and it should be tested for before selecting the water sources to be used in a water supply scheme, and also at regular intervals during the operation of the scheme. In most cases water with a poor bacteriological quality or whose bacteriological quality deteriorates is being contaminated by human or animal wastes. These wastes are dangerous as they are responsible for carriage or habitation of pathogens. Bacteriological quality should not be confused with aesthetically pleasing water. Aesthetically pleasant waters may not necessarily meet bacteriological quality standards. All water irrespective of how pleasant it appears should be tested to ascertain its bacteriological quality. High bacteriological quality standards are best achieved by selecting a water source which is not contaminated or which is located far away from sources of contamination.

5.5.2 Guidelines for Raw Water Treatment

Determination of the bacteriological quality of water is done by use of indicator organisms. Indicator organisms can be used for a range of purposes:

i) As indicators of faecal pollution in verification and surveillance monitoring;
ii) As indicators of the effectiveness of treatment processes such as filtration or disinfection in validation; and
iii) To show Integrity and cleanliness of distribution systems in operational monitoring.

The commonly used indicator organism is the coliform bacteria. Coliforms are bacteria that are always present in the digestive tracts of warm blooded animals, including humans, and are found in their wastes. They can also be found in plant and soil material. The most basic test for bacterial contamination of a water supply is the test for total coliform bacteria. Total coliform counts give a general indication of the sanitary condition of a water supply. Different species of coliform bacteria exist and their presence in water gives information about the possible origin of contamination.

The procedures for determining presence of *E. coli* include membrane filtration followed by incubation of the membranes on selective media at 44–45 °C and counting of colonies after 24 hours. Alternative methods include most probable number (MPN) procedures using tubes or micro titre plates and presence/absence tests, some for volumes of water larger than 100 ml. Field test kits are available for preliminary investigations only.

Since the presence of E. coli is an indication of recent faecal contamination, further action should be taken on detection. Such actions may include further sampling, investigation of potential sources such as inadequate treatment or breaches in distribution system integrity. With respect to the bacteriological quality, the following types of microorganisms shall strictly be ensured to be absent in drinking water or bottled (containerised) water; coliform in 250 ml, E. coli in 250 ml, Staphylococcus aureus in 250 ml, sulphite reducing anaerobes in 50 ml, Streptococcus faecalis, Shigella in 250 ml, Pseudomonas aeruginosa fluorescence in 250 ml and Salmonella in 250 ml. For drinking and containerised water, the max of the total viable counts at 37 °C per ml shall be 100 and 20 respectively.

Water quality analysis tests for determination of bacteriological water quality are to be carried out by qualified water quality analysts. These tests can be carried out at the National Water Quality Analytical laboratories of the MWE, National Water and Sewerage Corporation Water Analytical laboratories, Ministry of Internal Affairs, Government Chemist and at Universities with water Quality Analytical Laboratories.

Established standard methods for analysis of water and wastewater quality such as those of ISO or other reliable methods may be adopted for routine examinations.

The publication “Standard Methods for the Examination of Water and Wastewater” 17th Edition (1989), published by the American Public Health Association (APHA), American Water Works Association (AWWA) and the Water Pollution Control Federation (WPCF) is highly recommended for water quality analysis. Drinking water shall conform to the maximum microbiological limits given in Table 5-2

Table 5-2 Guidelines for Treatment Required for Raw Water

Coliform Organisms (Number/ 100 ml)	Treatment Required
0 - 50	Bacterial quality requiring disinfection only
50 - 5000	Bacterial quality requiring full treatment (coagulation, flocculation, sedimentation, filtration and disinfection)
5000 - 50000	Heavy pollution requiring extensive treatment
Greater than 50000	Very heavy pollution unacceptable as source unless no alternative exists. Special treatment required

When more than 40% of the numbers of coliforms are found to be of the Faecal Coliform Group, the water source should be considered to fall into the next higher category with respect to the treatment required.

5.5.3 Guidelines for Distributed Water

Water in the distribution system, whether treated or not, should not contain any organisms which may be of faecal origin. The absence of organisms of the coliform group, as defined in Table 5-2 should be considered as a fairly reliable indication of absence of pollution. Their presence should be assumed to be due to faecal pollution unless their non-faecal origin can be proved. The presence of E. coli is an indication of recent faecal pollution and it should prompt further investigation to determine their source. However, water quality can vary rapidly, and all systems are at risk of occasional failure. For example, rainfall can greatly increase the levels of microbial contamination in source waters, and waterborne outbreaks often occur following rainfall. Results of analytical testing must be interpreted taking this into account. Chemical disinfection of a drinking-water supply that is contaminated by faecal matter will reduce the overall risk of disease but may not necessarily render the supply safe (WHO Report, 2011).

If deficiencies in water quality are detected, remedial action has to be taken promptly. Such action may be temporary, like special campaigns to promote the boiling of drinking water in homes, and/or long term measures for localizing and eliminating the source of contamination and improving treatment.

5.6 Chemical Quality

5.6.1 Constituents of Health Significance

Maximum limits for chemicals in water are derived from the health risks associated with consumption of these chemical if found in water. Drinking water should conform to the maximum limits for chemicals of health significance as indicated in Appendix 3.

5.6.2 Chemical Parameters Indirectly Related to Health

Phosphorous and Nitrogen compounds, such as Nitrite and Ammonium are indirectly related to health. The two natural elements (also referred as nutrients) namely Nitrogen (N) and Phosphorus (P) are essential for plant and animal growth, maintenance and reproduction. Excessive Nitrate (NO_3) in drinking water can cause human and animal health problems, particularly for small babies. Excess Nitrogen and Phosphorus in surface waters and Nitrogen in groundwater cause eutrophication (excessive algal growth leading to an algal bloom) in surface waters and health problems in humans and livestock as a result of high intake of Nitrogen in its Nitrate form.

The effects of eutrophication on the environment may have deleterious consequences for the health of exposed animal and human populations through various pathways. Specific health risks appear when fresh water, extracted from eutrophic areas is used for the production of drinking water. Severe impacts can also occur during animal watering in eutrophic water. The guideline values given in Table 5-3 should not be exceeded in drinking water

Table 5-3: Chemical Parameters Indirectly Related to Health

Parameter	Unit	Guideline Value
Nitrite, NO_2	mg/l	1.0
Ammonium, NH_4	mg/l	1.0
Phosphorus (as Phosphate, PO_3)	mg/l	0.5

5.6.3 Permissible Aesthetic Quality

Permissible can be taken to mean the acceptable upper limit which can still render the usage of the water safe. Desirable is the planned (wishful) limit that is worth achieving the portability of the water. Under circumstances where it is not practicable to supply water of the “desirable” aesthetic quality, it may be permissible to adopt the guideline values given in Table 5-4.

Table 5-4: Permissible Aesthetic Quality

PARAMETER	UNIT	GUIDELINE VALUE
Chloride	mg/l	600
Colour	TCU	50
Copper	mg/l	1.5
Iron	mg/l	1.0
Manganese	mg/l	0.5
pH	-	6.5-9.2
Solids	mg/l	1500
Turbidity	NTU	25
Zinc	mg/l	15

5.6.4 Parameters Affecting Organoleptic and Physical Characteristics

Common constituents that in concentrations normally present in water, do not affect health may however affect the aesthetic quality and physical characteristics of the water. If these are present in excess they may render the water not suitable for use, appearance and taste. The Uganda Standard US 201: 2008 specifies the water requirements for these parameters. The quality of water should comply with the US 201: 2008 or WHO requirements indicated in Appendix 3).

5.6.5 Constituents Affecting Building and Pipe Materials

5.6.5.1 Introduction

Materials commonly used in the construction of water supply schemes are cement products, steel, iron and plastic. Plastic is generally unaffected by water. In general, substances aggressive to the construction materials have to be removed from water or alternative construction materials resistant to the “aggressivity” of water have to be used. In this regard it should be appreciated that the aggressive attacks can be from the inside or the outside of the construction. For example, external "aggressivity" can be caused by groundwater, swampy ground or humic acid soils, especially peaty soils and those containing Calcium Sulphate. Corrosion caused by aggressive water, leads not only to the loss of pipes and building materials, but also to a reduction in water quality.

Carbon dioxide is soluble in water, in which it reversibly converts to H_2CO_3 (carbonic acid). The relative concentrations of CO_2, H_2CO_3, HCO_3^- (bicarbonate) and CO_3^{2-} (carbonate) ions depend on the pH of the water. In neutral or slightly alkaline water (pH > 6.5), the bicarbonate (HCO_3^-) predominates becoming the most prevalent (> 95%). In very alkaline water (pH > 10.4), the predominant form is carbonate (CO_3^{2-}). As long as there remain Bicarbonates in the water, the buffering capacity remains intact. In areas rich in limestone ($CaCO_3$), aquatic systems are thus very well buffered against changes in pH.

The pH of pure water is 7. This makes pH value a good indicator of whether water is hard or soft. Generally, water with pH lower than 7 is acidic, and with pH greater than 7 is basic; pHs values higher than 9 or lower than 3 are lethal to many organisms. Whereas the pH range for surface water systems is 6.5 to 8.5, the pH range for groundwater systems is 6 to 8.5. Alkalinity is a measure of the capacity of the water to resist a change in pH that would tend to make the water more acidic. The measurement of alkalinity and pH is needed to determine the corrosiveness of the water. In general, water with a pH < 6.5 could be acidic, soft, and corrosive. Acidic water could contain metal ions such as iron, manganese, copper, lead, and zinc. In other words, acidic water contains elevated levels of toxic metals. Acidic water can cause premature damage to metal piping, and have associated aesthetic problems such as a metallic or sour taste. It can also stain laundry and cause "blue-green" colour staining on sinks and drains.

Fresh water can have widely ranging pH values depending on the geology of the drainage basin or aquifer and the influence of contaminant inputs (acid rain). pH value can also drop due to treatment processes like addition of alum. If the water is acidic (lower than 7), lime, soda ash, or sodium hydroxide can be added to raise the pH during water purification processes. Lime addition increases the calcium ion concentration, thus raising the water hardness. This process is called "pH correction". For highly acidic waters, forced draft degasifiers[3]can be an effective way to raise the pH, by stripping dissolved carbon dioxide from the water. Making the water alkaline helps coagulation and flocculation processes work effectively and also helps to minimize the risk of lead being dissolved from lead pipes and from lead solder in pipe fittings. Sufficient alkalinity also reduces the corrosiveness of water to iron pipes. Acid (carbonic acid, hydrochloric acid or sulfuric acid) may be added to alkaline waters in some circumstances to lower the pH. Alkaline water (above pH 7.0) does not necessarily mean that lead or copper from the plumbing system will not be dissolved into the water. The ability of water to precipitate calcium carbonate to protect metal surfaces and reduce the likelihood of toxic metals being dissolved in water is a function of pH, mineral content, temperature, alkalinity and calcium concentration.

5.6.5.2 Concrete

Acid water with pH values falling below those given in Table 5-5will have a damaging effect on concrete.

[3]Degasification is the removal of dissolved gases from liquids, especially water or aqueous solutions

Table 5-5: pH Values Related to Concrete Damage

Hardness $CaCO_3$, mg/l	pH
50	8.4
100	8.1
150	7.8
200	7.5
250	7.3
300	7.1

Soft water (with low carbonate hardness) becomes very aggressive if it contains free carbon dioxide. This aggressive carbon dioxide dissolves the calcium salts of concrete and mortar, gradually destroying them. If such water is also flowing, the rate of destruction can be quite rapid. Alkaline water with pH values above those given in Table 5-5, can also cause damage to cement products if the sulphate content is above 300 mg/l in stagnant water or 100 mg/l in flowing water. Calcium and Magnesium Sulphates and, to a small extent, the corresponding chlorides, can also destroy concrete.

Water containing hydrogen sulphide and large amounts of ammonium salts caused by wastes, is also damaging to concrete. Concrete is also attacked by water containing sodium hydrogen carbonate. Large amounts of aggressive carbon dioxide occurring with low carbonate hardness will cause damage to asbestos cement pipes. Internal bitumen linings and external coal-tar coats, can improve the resistance of asbestos cement pipes quite considerably.

5.6.5.3 Steel and Iron Products

Stagnant water causes greater corrosion to steel and iron pipes than flowing water. Therefore, aggressive water is particularly damaging to the terminal components of pipeline systems.

Water with no aggressive carbon dioxide, flowing at less than 0.5 m/s, having a hardness exceeding 35 mg/l and a dissolved oxygen content exceeding 6 mg/l forms a protective anti-rust layer of calcium and magnesium compounds on the internal surfaces of pipes. If the flow velocity of the water is more than 0.5 m/s, the same will occur if the dissolved oxygen content is more than 2 mg/l.

If the oxygen content is too low, all types of water will attack steel and iron pipes, even if all the other corrosion factors do not favour attack. The oxygen concentration should, therefore, never be less than 4.0 mg/l even if the velocity of flow exceeds 0.5 m/s.

Iron and steel are always attacked and dissolved by water containing aggressive carbon dioxide which prevents the formation of the protective anti-rust layer. For iron pipes, the water pH values should always be equal to or just below those given in Table 5-5. For galvanized steel pipes, the pH values of the water should be kept at a level of 0.5 pH units below the values given in Table 5-5.

Unprotected iron pipes are attacked by hydrogen sulphide. Water with a high chloride content attacks iron pipes virulently. The limit for unprotected iron pipes is 150 mg/l in soft waters. Steel pipes are more susceptible to chemical attacks than cast iron pipes, especially when exposed to soft aggressive water. For this water, steel and iron pipes should be avoided in favour of uPVC pipes and special protective coating should be considered for concrete storage reservoirs.

5.6.5.4 Galvanic Corrosion

Galvanic or bi-metallic corrosion is a common form of corrosion. It occurs when two different metals are connected and immersed in a solution such as water. In these circumstances, a set of galvanic cells is formed in which corrosion is caused by the electrochemical action between the two metals. An example of this type of corrosion is a well screen made of two different metals, such as mild steel and stainless steel.

There are three conditions in the absence of which galvanic corrosion fails to occur.

i) First there must be two electrochemically dissimilar metals present;
ii) Second, there must be an electrically conductive path between the two metals; and
iii) There must be a conductive path for the metal ions to move from the more anodic metal to the more cathodic metal.

Galvanic corrosions may be prevented in a number of ways:

i) Electrically insulate from each other metals from different groups, wherever practical. If complete insulation cannot be achieved, paint or plastic coating at joints will help;
ii) Avoid making combinations where the area of the less noble, anodic metal is relatively small compared with the area of the more noble metal;
iii) Select combinations of metals which will be in electrical contact from groups as close together as possible in the galvanic series;
iv) If you must use dissimilar materials well apart in the series, avoid joining them by threaded connections as the threads will probably deteriorate excessively. Brazed or thermal joints are preferred, using a brazing alloy more noble than at least one of the metals to be joined;
v) Apply coatings with judgment. Example: Do not paint the less noble metal without also painting the nobler; otherwise, greatly accelerated attack may be concentrated at imperfections in coatings on the less noble metal. Keep such coatings in good repair; and
vi) Consider use of cathodic protection.

5.6.6 Organoleptic and Physical Standards

Drinking water shall conform to the US 201:2008 requirements for factors affecting organoleptic and physical characteristics corresponding to a given class indicated in Appendix 3.

5.6.7 Chemical standards

Drinking water shall conform to the maximum limits for chemicals of health significance in drinking water as indicated in Appendix 3. It should be noted that, safety factors indicated in Appendix 3 apply to both Class I and Class II.

5.6.8 Water Quality Preservation and Monitoring

5.6.8.1 General

All water supply schemes should be designed to preserve water quality. The most important step in meeting this requirement is to site all system facilities in such a way as to minimize the risk of contamination. For example, water sources should be provided with appropriate protection zones as already discussed in Chapter 3 – “Water Sources”. Design criteria and installations for preserving water quality are detailed in the relevant sections throughout this design manual.

The monitoring of drinking water quality is not only a matter of establishing regulations but it is also a matter of establishing the necessary supportive related physical facilities including laboratories, infrastructure, organizational as well as educational measures. Each water supply scheme should have a water quality monitoring strategy and plan that details the parameters checked, the frequency of testing and the location of sampling points. The results of every water quality test must be kept as a record for inspection.Water supply schemes should be designed in such a way as to facilitate water quality monitoring, so that water samples can be readily taken especially at the following points:

i) Water sources, before treatment;
ii) Water treatment stages (at entry and exit of every unit operation) to check efficiency of treatment process; and
iii) Different points in the transmission and distribution systems.
iv) Consumer draw points including stand pipes, individual connections and reservoirs.

The general and sampling requirements for water quality surveillance can be found in the US 201:2008. The list of the water quality monitoring stations for the different parts of Uganda can be found in Appendix 3.

5.6.8.2 Water Safety Plans

In order to ensure long term safety of the water supplied, the designer should give guidance on the possible plans to maintain the quality of water.

A water safety plan shall consist of three key components:

i. System assessment to determine whether the drinking-water supply chain (up to the point of consumption) as a whole can deliver water of a quality that meets health-based targets;
ii. Identifying control measures in a drinking water system that will collectively control identified risks and ensure that the health-based targets are met; and
iii. Management plans describing actions to be taken during normal operation or incident conditions and documenting the system assessment (including upgrade and improvement), monitoring and communication plans and supporting programmes.

A water safety plan shall include:

i) Measures to protect the source of drinking water from risks of pollution;
ii) Measures to ensure all installations intended for the production of drinking water exclude any possibility of contamination. For this purpose and in particular:
    - The installation for collection, the pipes and the reservoirs shall be made from materials suited to the water and in such a way as to prevent the introduction of foreign substances in water;
    - The equipment and its use for production, especially installation for washing and packaging, shall meet hygienic requirements;
iii) Measures to ensure an appropriate treatment such as pre-treatment coagulation, flocculation, sedimentation, filtration and disinfection are undertaken to assure the safety of water for the consumers;
iv) Appropriate operational monitoring system including monitoring parameters that can be measured and for which limits have been set to define the operational effectiveness of the activity; frequency of monitoring and procedures for corrective action that can be implemented in response to deviation from limits. If, during production it is found that the water is polluted, the producer shall stop all operations until the cause of pollution is eliminated; and
v) A verification plan to ensure that individual components of a drinking-water system, and system as a whole is operating safely.

5.6.8.3 Surveillance and Sampling Requirements

Drinking water suppliers should ensure, at all times, the quality and safety of the water that they produce (US 201:2008). Public health surveillance (that is, surveillance of health status and trends) contributes to verifying drinking-water safety. Adequate infrastructure, proper monitoring and effective planning and management and a system of independent surveillance are basic and essential requirements to ensure the safety of drinking water. Surveillance shall cover the total water supply network from the source of untreated water to the consumer delivery points.

A sampling programme that takes into consideration appropriate international recommendations shall be established and implemented. The sampling shall be regular and its frequency shall mainly depend on the following factors:

i) Quality of water harnessed including effects on the water from climatic, human and industrial activities;
ii) Type of treatment for drinking worthiness;
iii) Volume of water processed;
iv) Risks of contamination;
v) Background of public water supply network;
vi) Population served; and
vii) Capabilities of the analytical facility (both in terms of capacity and in terms of analytical performance).

With respect to sampling requirements, the recommendations given in US ISO 5667-1 shall be used as the basis for the establishment of a sampling programme, and the recommendations given in US ISO 5667-2, US ISO 5667-3, US ISO 5667-4, US ISO 5667-5, US ISO 5667-6 and US ISO 5667-11 shall be used as the basis for implementing the sampling programme.

A sampling program for the different categories of water supplies should be established and adhered to. In all cases a minimum sampling frequency given in Table 5-6 shall be used as a guide where no established program is available.

Table 5-6: Minimum Frequency of Sampling of Water for Surveillance

Population Served	Frequency * (Minimum) of Sampling
More than 100 000	10 samples every month per 100,000 of population served
25 001 – 100 000	10 samples every month
10 001 – 25 000	3 samples every month
2 500 – 10 000	2 samples every month
Less than 2 500	1 sample every month
* During the rainy season, sampling should be carried out more frequently	

5.6.8.4 Parameters Required for Minimum Monitoring

Due to the fact that the cost of performing a full analysis against the various water quality parameters listed in the various tables of Appendix 3, the analysis of the parameters in appendix 3) may be deemed acceptable for the purpose of indicating on going levels of operational efficiency in a water treatment plant.

5.6.8.5 Water Sampling

The selection of water sources and treatment methods will require the collection and analysisof water samples from the alternative sources available for water supply. The samples taken should cover all regimes of the water source under investigation and they should be taken in sufficient numbers covering the dry and rainy seasons. Samples from raw water wells and boreholes should be taken after at least 24 hours of pumping. The results of the water sample analyses should be evaluated against the water quality guidelines. Whenever the results leave some doubts regarding the selection of water sources and/or treatment methods, additional samples should be collected and analyzed.

Water sampling and analysis should be carried out by trained and skilled personnel as incorrect techniques can lead to incorrect results and decisions. The Water Quality Management Strategy, 2006 of the MWE gives a guide on sampling and its frequency and number of samples. The strategy recommends tests to be done on parameters specified in Appendix 3.

The sampling points, samples must be taken from locations that are representative of the water source, treatment plant, storage facilities, distribution network, points at which water is delivered to the consumer, and points of use (WHO, 2010). In selecting sampling points, each locality should be considered individually however, the following general criteria are usually applicable:

i) Sampling points should be selected such that the samples taken are representative of the different sources from which water is obtained by the public or enters the system;
ii) These points should include those that yield samples representative of the conditions at the most unfavourable sources or places in the supply system, particularly points of possible contamination such as unprotected sources, loops, reservoirs, low-pressure zones, ends of the system, etc;
iii) Sampling points should be uniformly distributed throughout a piped distribution system, taking population distribution into account;
iv) The number of sampling points should be proportional to the number of links or branches;
v) The points chosen should generally yield samples that are representative of the system as a whole and of its main components;
vi) Sampling points should be located in such a way that water can be sampled from reserve tanks and reservoirs, etc;
vii) In systems with more than one water source, the locations of the sampling points should take account of the number of inhabitants served by each source; and
viii) There should be at least one sampling point directly after the clean-water outlet from each treatment plant.

Sampling sites in a piped distribution network may be classified as:
i) fixed and agreed with the supply agency;
ii) fixed, but not agreed with the supply agency; or
iii) random or variable.

Each type of sampling site has certain advantages and disadvantages. Fixed sites agreed with the supplier are essential when legal action is to be used as a means of ensuring improvement; otherwise, the supply agency may object to a sample result on the grounds that water quality may have deteriorated in the household, beyond the area of responsibility of the supplier. Nevertheless, fixed sample points are rare or unknown in some countries.

Fixed sites that are not necessarily recognized by the supply agency are used frequently in investigations, including surveillance. They are especially useful when results have to be compared over time, but they limit the possibility of identifying local problems such as cross-connections and contamination from leaking distribution networks. Sampling regimes using variable or random sites have the advantage of being more likely to detect local problems but are less useful for analysing changes over time.

5.6.8.6 Handling of Water Samples

Water samples should be handled with maximum care as poor handling may lead to an erroneous test result. All samples must be collected using methods that ensure sterility of the utensils used for collection. Caution should be exercised as a water source that is not contaminated can be contaminated by the utensil used or by the person collecting the sample.

The time between sample collection and analysis should, in general, not exceed 6 hours, and 24 hours is considered the absolute maximum (WHO, 2010). Samples should be immediately placed in a lightproof insulated box containing melting ice or ice-packs with water to ensure rapid cooling. If ice is not available, the transportation time must not exceed 2 hours. It is imperative that samples are kept in the dark and that cooling is rapid. If these conditions are not met, the samples should be discarded. When water that contains or may contain even traces of chlorine is sampled, the chlorine must be inactivated. If it is not, microbes may be killed during transit and an erroneous result will be obtained. The bottles in which the samples are placed should therefore contain sodium thiosulfate to neutralize any chlorine present.

Supporting documents cited in Annex 1 of WHO (2011) are recommended in the implementation of water safety plan. Table 5-7 showing the ISO standards cited in WHO (2011) shall be adopted water sampling, sampling design, sample handling and preservation.

Table 5-7: International Organization for Standardization (ISO) Standards for Water Quality Giving Guidance on Sampling [a]

ISO standard No.	Title (water quality)
5667-1:2006	Sampling—Part 1: Guidance on the design of sampling programs and sampling techniques
5667-3:2003	Sampling—Part 3: Guidance on the preservation and handling of water samples
5667-4:1987	Sampling—Part 4: Guidance on sampling from lakes, natural and man-made
5667-5:2006	Sampling—Part 5: Guidance on sampling of drinking water and water from treatment works and piped distribution systems
5667-6:2005	Sampling—Part 6: Guidance on sampling of rivers and streams
5667-11:2009	Sampling—Part 11: Guidance on sampling of ground waters
5667-13:1997	Sampling—Part 13: Guidance on sampling of sludge from sewage and water treatment works
5667-14:1998	Sampling—Part 14: Guidance on quality assurance of environmental water sampling and handling
5667-16:1998	Sampling—Part 16: Guidance on bio-testing of samples
5667-20:2008	Sampling—Part 20: Guidance on the use of sampling data for decision making—Compliance with thresholds and classification systems
5667-21:2010	Sampling—Part 21: Guidance on sampling of drinking water distributed by tankers or means other than distribution pipes
5667-23:2011	Sampling—Part 23: Guidance on passive sampling in surface waters
5668-17:2008	Sampling—Part 17: Guidance on sampling of bulk suspended sediments
13530:2009	Guidance on analytical quality control for chemical and physicochemical water analysis
17381:2003	Selection and application of ready-to-use test kit methods in water analysis

[a] ISO has also established quality management standards relating to drinking-water supply, including ISO 24510:2007.
Activities relating to drinking water and wastewater services—Guidelines for the assessment and for the improvement of the service to users; and ISO 24512:2007.
Activities relating to drinking water and wastewater services—Guidelines for the management of drinking water utilities and for the assessment of drinking water services.

Source: WHO, 2011

Chapter 6

WATER TREATMENT

6.1 General

6.1.1 Basic Considerations

In most cases, it will be necessary to treat water in order to render it fit for drinking and other domestic uses. Most important in water treatment is the removal of pathogenic organisms and toxic substances such as heavy metals, which may cause health hazards. Other substances may also need to be greatly reduced if not completely removed. Such substances include suspended matter (which causes turbidity); iron and manganese compounds which impart bitter taste or staining of laundry; excessive carbon dioxide which corrodes concrete and metals; hardness; total dissolved solids and organic matter. The water quality guidelines presented in the Chapter 5 "Water Quality" should be used when deciding on the types and extent of treatment required.

Before a treatment plant is designed, thorough investigations should be done to determine whether it would not be feasible to use alternative sources with better raw water quality. Various water treatment processes have been developed, and often a treatment result can be achieved in a number of different ways as illustrated in Table 6-1.

Table 6-1: Effectiveness of Water Treatment Processes in Removing Impurities

Water Quality Parameter	TREATMENT PROCESSES					
	Aeration	Chemical Coagulation and Flocculation	Sedimentation	Rapid Filtration	Slow Sand Filtration	Final Disinfection
Dissolved Oxygen	+	o	o	-	-	+
Carbon Dioxide Removal	+	o	o	+	++	+
Turbidity Reduction	o	+++	+	+++	++++	o
Colour Reduction	o	++	+	+	++	++
Taste and Odour Removal	++	+	+	++	++	+
Bacterial Removal	o	+	++	++	++++	++++
Iron and Manganese Removal	++	+	+	++++	++++	o
Organic Matter Removal	+	+	++	+++	++++	+++

Notes:++++ = increasing positive effect   o = no effect   - =negative effect

Turbidity in water is caused by the presence of suspended matter, scattering and absorbing light rays and thus giving the water a non-transparent milky appearance. Common treatment processes for groundwater and surface water are summarized in Table 6-2 and Table 6-3 respectively.

Table 6-2: Treatment of Groundwater

Water Quality	TREATMENT PROCESSES				
	Aeration for increasing O_2	Aeration for reducing CO_2	Plain Sedimentation	(Rapid) Filtration	Final Disinfection
Aerobic, fairly hard, not corrosive water					x
Aerobic, soft, corrosive water		x			x
Anaerobic, fairly hard, not corrosive, no iron and manganese	x				x
Anaerobic, fairly hard, not corrosive with iron and manganese	x		o	x	x
Anaerobic, soft, corrosive, no iron and manganese	x	x			x
Anaerobic, soft, corrosive with iron and manganese	x	x	o	x	x

[x = necessary treatment process, o = optional treatment process]

Table 6-3: Treatment of Surface Water

Water Quality	TREATMENT PROCESSES					
	Pre-chlorination	Chemical Coagulation and Sedimentation	Sedimentation	Rapid Filtration	Slow Sand Filtration	Final Disinfection
Clear and unpolluted						x
Slightly polluted, low turbidity				o	x	x
Slightly polluted, medium turbidity			o	x	x	x

Water Quality	TREATMENT PROCESSES					
	Pre-chlorination	Chemical Coagulation and Sedimentation	Sedimentation	Rapid Filtration	Slow Sand Filtration	Final Disinfection
Slightly polluted, high turbidity		x	x	x	x	x
Slightly polluted, many algae	x	x	x	x		x
Heavily polluted, little turbidity	x			x	x	x
Heavily polluted, high turbidity	x	x	x	x		x

[x = necessary treatment process, o = optional treatment process]

6.1.2 Large Towns and Urban Centres

Water treatment in large towns and urban centres should be effective enough to achieve the following:

i) The bacteriological standards as set out in Chapter 5 – "Water Quality";
ii) Chemical standards as set out in Chapter 5 – "Water Quality"; and
iii) Produce water which complies with the requirements set out in Chapter 5 – "Water Quality".

6.1.3 Rural Areas

Water treatment in rural areas should be effective enough to achieve the following:

i) The bacteriological standards as set out in Chapter 5 – "Water Quality";
ii) The chemical standards as set out Chapter 5 – "Water Quality"; and
iii) Produce water which complies with the requirement set out in Chapter 5 – "Water Quality".

6.1.4 General Design Principles

The following general principles should be observed in the planning and design of water treatment works:

i) The works should be designed for continuous 24-hour operation;
ii) All major process units (chemical feeders, mixing basins, flocculation tanks, sedimentation tanks and filters) which require frequent servicing or cleaning should be provided at least in duplicate. Sedimentation tanks and filters should preferably be provided in triplicate, in order to limit the temporary overloading when one unit is being serviced or cleaned to 50%;
iii) However, in small treatment works (of capacity less than 500 m3/day), it may not be practicable to have process units in duplicate or triplicate as they will then be too small; and

iv) As a general principle, all kinds of electrical, mechanical and automatic equipment is to be kept to the absolute minimum possible.

6.1.5 Phasing of Water Treatment Works

Even when water analyses do not indicate immediate or future need for treatment, provisions should always be made to incorporate full treatment at a later stage, to cater for deteriorating raw water quality or increased demand for high-quality potable water. In this regard, suitable sites for the treatment works should be identified and adequate gradients provided to allow for the additional head losses through the works. There should also be provision of adequate space for future extensions and phases of the treatment works.

Units like flocculation tanks which depend on the flow velocity of the water for their effective performance, have to be designed in such a way that they can function properly also for flows smaller than the "Ultimate Year" flows.

6.2 Raw Water Storage

6.2.1 General

Raw water storage can be regarded as a form of treatment process. A raw water storage basin can serve the following purpose:

i) Reducing turbidity by sedimentation;
ii) Reducing the numbers of pathogenic bacteria through the activity of algae, other organisms and the ultra-violet rays in sunlight; and
iii) Facilitating the destruction of other water-based pathogenic micro-organisms such as schistosomiasis cercariae, for which a storage period of 48 hours is sufficient.

However, prolonged storage of raw water can cause the following problems:

i) Reduction of the oxygen content of the water in which case aeration may be required as the first treatment process;
ii) Promotion of algal growth in the water; and
iii) Loss of water through evaporation and seepage.

6.2.2 Design

A raw water storage basin can be made by constructing simple earth dykes up to a height of about 6 m. The dead storage allowance should be about 2 m and the detention time should be of the order of several weeks to a few months. Allowance should also be made for losses due to evaporation and seepage which could be as high as 15-25 mm/day. Adequate hydrological study is required to ensure that the storage is sufficient to cover requirements all year round, including for the dry seasons.

6.2.3 Infiltration

Artificial infiltration can improve the physical, chemical and bacteriological quality of surface water as already discussed in Chapter 3 – "Water Sources".

6.2.4 Plain Sedimentation

6.2.4.1 General

Plain sedimentation basins mainly serve the purpose of reducing turbidity by the removal of suspended matter. The basins may serve as a pre-treatment measure or as the only main treatment process where water quality requirements are not stringent. A plain sedimentation tank may have a batch wise or continuous operation and it can be constructed out of concrete, masonry or it can be a simple excavation with a protective lining. Plain sedimentation can be used as a pre-treatment stage upstream of slow sand filters, if the average annual turbidity of the water is in the range of 20 – 100 NTU.

6.2.4.2 Design

Plain sedimentation tanks should be designed for a surface loading in the range of 0.1 - 0.5 m^3/m^2h. The exact surface loading to be adopted should be determined after carrying out settlement tests on samples of raw water, typical of all regimes of the water source. The settling properties of water will depend on the soil and vegetation conditions in the catchment area, and they will vary considerably between different locations and regimes of the water source.

6.2.5 Roughing Filtration

6.2.5.1 General

Roughing filtration can be used as a pre-treatment process upstream of sand filtersas a process for reducing turbidity. It has also a useful part to play in many treatment processes. In a typical roughing filter, there are series of tanks which are filled with progressively smaller diameter media in the direction of the flow which can either be horizontal or vertical. With respect to the vertical flow direction, we have either up-flow or down-flow roughing filters.

The media in the tank which may include gravel, rice husks or any other suitable local material, plays an important role of reducing the vertical settling distance of the particles to a distance of a few millimetres. The tendency of the sediments building up on the media leads to its sloughing off and eventual accumulation in the lower section while simultaneously regenerating the upper portions of the filter; thus, periodic cleaning is required as a maintenance process to remove the accumulated silt. For the detail information on the design of roughing filters, designers are recommended to read a comprehensive SANDTEC design report entitled “Surface Water Treatment by Roughing Filters - A Design, Construction and Operation Manual, 1996”.

6.2.5.2 Up-Flow Filters

Up-flow filters are often suitable as roughing filters, as they provide for a coarse - to - fine filtration process and are thus not liable to rapid clogging.

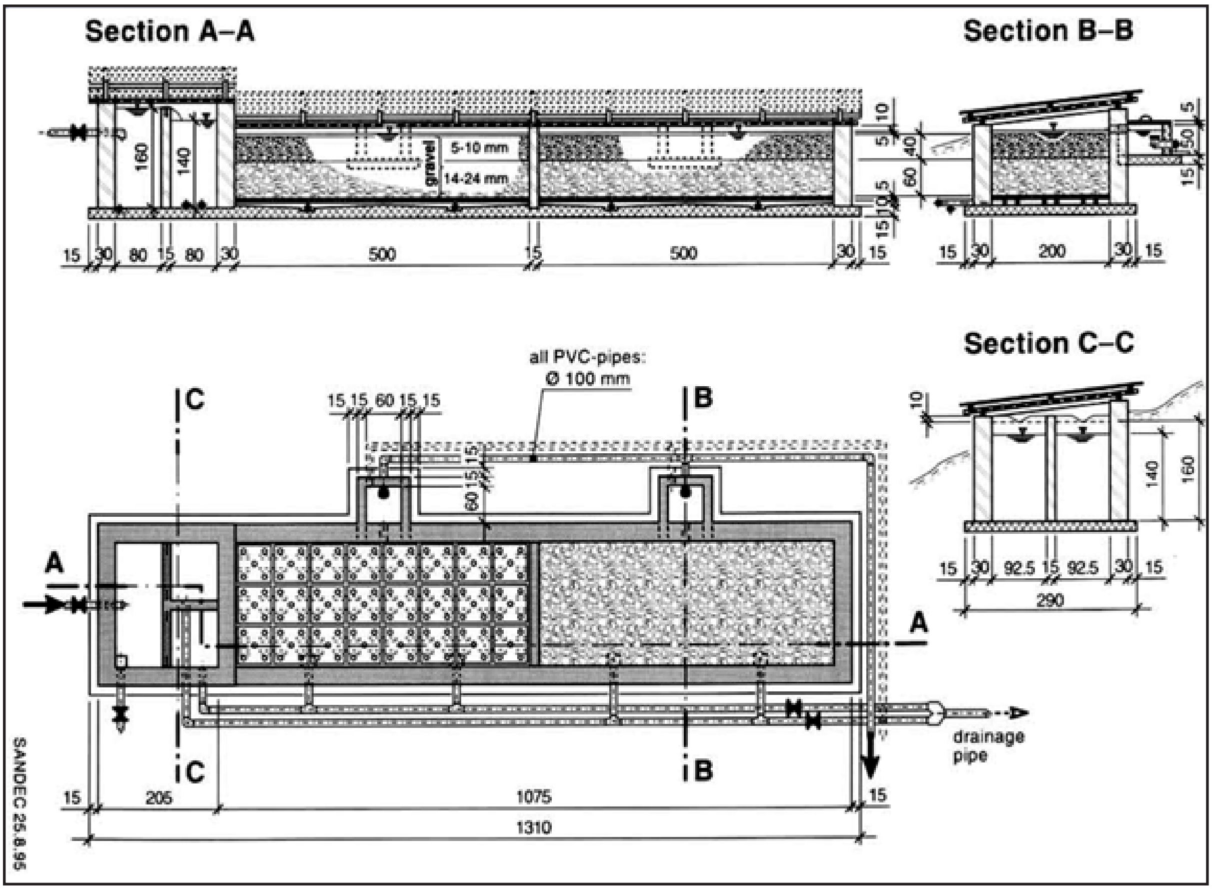


Figure 6-1: Example of an Upflow Roughing Filter.
Source: SANDTEC – SKAT, 1996.

The backwash rate can be relatively low as no expansion of the filter bed is needed, but the washing takes a long time of about 20 - 30 minutes. Raw water can be used for backwashing. A typical up-flow filter is given in Figure 6-1below.

6.2.5.3 Horizontal Gravel Filters

Horizontal gravel filters should have a depth of 1 - 2 m, sub-divided into three zones each about 5 m long and consisting of gravel of sizes 20 - 30 mm at the inlet side, followed by sizes 15 - 20 mm in the middle and sizes 10 - 15 mm at the outlet end. The horizontal water flow rate computed over the full depth should be in the range of 0.5 - 1.0 m/h, which represents a surface loading of 0.03 - 0.1 m^3/m^2h. Cleaning of the gravel will be needed only after a period of a number of years. Turbidity removals of 60 - 70% can be achieved for waters having turbidity of up to 150 NTU.

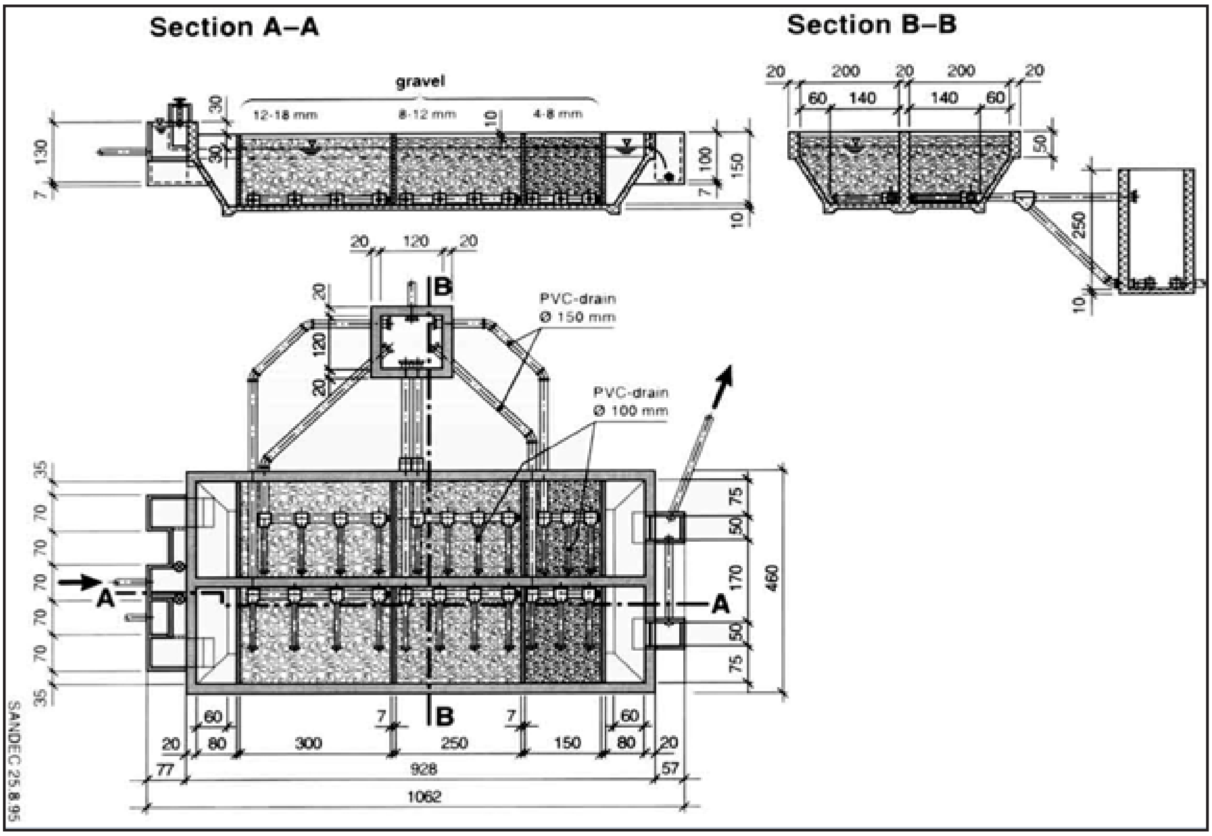


Figure 6-2: Example of a Horizontal Flow Roughing Filter.
Source: SANDTEC – SKAT, 1996.

6.2.5.4 Rapid Roughing Sand Filters

Rapid roughing sand filters are built like conventional vertical - flow rapid sand filters, except that the sand should be coarser, with effective size of 0.8 - 1.2 mm. Filter backwashing can be done using raw water. A rapid roughing sand filter can be used as a pre-treatment process upstream of slow sand filters, if the turbidity of the raw water is in the range of 20 - 100 NTU. The filters should be designed for a surface loading in the range of 5 -15 m^3/m^2h.

6.3 Aeration

6.3.1 General

Aeration is the process whereby water is brought into intimate contact with air for the following purposes:

i) Increasing the oxygen content of the water;
ii) Reducing the carbon dioxide content of the water;
iii) Removing hydrogen sulphide, methane and other volatile organic compounds responsible for imparting tastes and odours; and
iv) Removing iron and manganese responsible for imparting tastes and discolouration.

6.3.2 Surface Water

Generally, there is no need for special provisions to be made for the aeration of surface water with the aim of reducing high contents of iron and/or manganese. The contents of iron and manganese as found in the surface waters in Uganda are usually in a form that can easily be removed by coagulation, flocculation, sedimentation and filtration[4]. However, aeration may be required to increase the oxygen content of waters abstracted from large storage reservoirs.

6.3.3 Groundwater

Aeration is widely used for the treatment of groundwater having unacceptably high contents of iron and/or manganese. The atmospheric oxygen brought into the water through aeration, reacts with the dissolved ferrous and manganous compounds, changing them into insoluble ferric and manganic oxide hydrates. The hydrates can then be removed by the subsequent processes of sedimentation and/or filtration.

6.3.4 Cascade Aerators

A cascade aerator consists of a flight of 4 - 6 steps, each about 300 mm high as illustrated in Figure 6-3[3]. To produce turbulence and thus enhance the aeration efficiency, obstacles are often sat at the edge of each step. The design capacity of a cascade aerator should be of the order of 35 m^3/hour per metre of width.

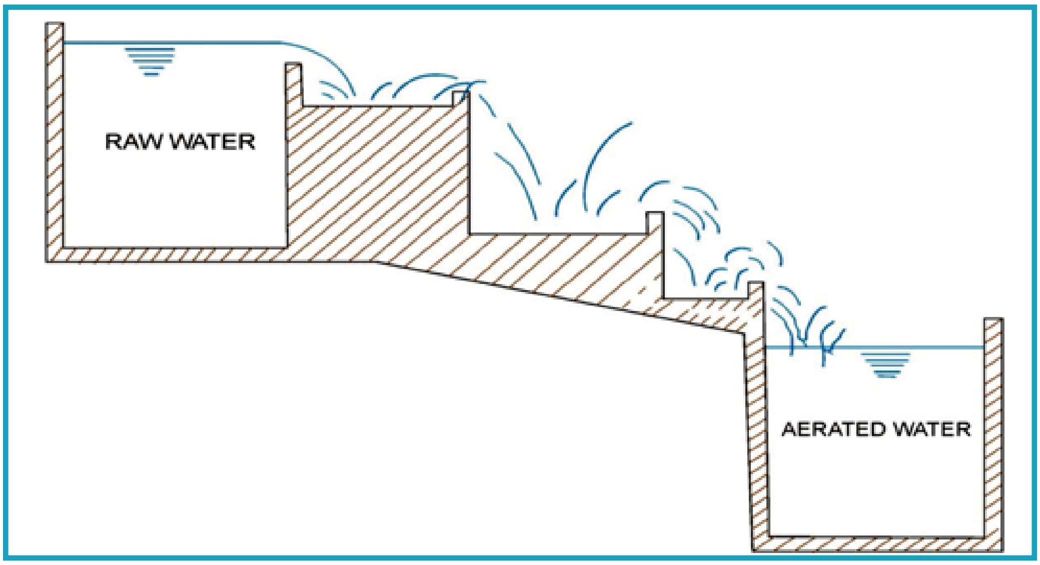


Figure 6-3: Cascade Aerator

6.3.5 Multiple Tray Aerators

A multiple tray aerator consists of 4 - 8 trays with perforated bottoms at intervals of 300 – 500 mm. Through perforated pipes, the water is divided evenly over the upper tray, from which it trickles down, the droplets being dispersed and re-collected at each successive tray. The design surface loading of a multiple tray aerator should be of the order of 70 $m^3/hour/m^2$.

6.4 Coagulation

6.4.1 General

Coagulation can be considered to be the first stage in the formation of precipitates of finely divided, suspended and colloidal matter in water, induced by the addition of special chemicals called “coagulants”. The commonest coagulant in Uganda is aluminium sulphate, commonly called alum. Other coagulants are polymers of various, often proprietary, types. Consideration should be made of the supply chain for polymers before they are for used.

[4]There are a few cases of high iron content found in swamp water which are mostly stagnant. Examples are found in Kayunga Town Water Supply, Pallisa Town Water Supply and Masaka Town Water Supply. Aeration helps in the removal of the high iron content; however, deposits of ferric hydrates over the aerators tend to leave the aerators discoloured and may also block the orifices.

6.4.2 Types of Coagulants

Coagulant chemicals are in two main types including primary coagulants and coagulant aids. Primary coagulants neutralize the electrical charges of particles in the water which causes the particles to clump together well as coagulant aids add density to slow-settling flocs and add toughness to the flocs so that they do not break up during the mixing and settling processes. Primary coagulants are always used in the coagulation/flocculation process while coagulant aids, in contrast, are not always required and are generally used to reduce flocculation time.

Chemically, coagulant chemicals are either metallic salts (such as alum) or polymers. Polymers are man-made organic compounds made up of a long chain of smaller molecules. Polymers can be cationic (positively charged), anionic (negatively charged) or nonionic (neutrally charged). Table 6-4 shows some of the common coagulant chemicals and lists whether they are used as primary coagulants or as coagulant aids.

Table 6-4 Types of Coagulants

Chemical Name	Chemical Formula	Primary Coagulant	Coagulant Aids
Aluminum sulfate (Alum)	$Al_2(SO_4)_3 \cdot 14\ H_2O$	X	
Ferrous sulfate	$FeSO_4 \cdot 7\ H_2O$	X	
Ferric sulfate	$Fe_2(SO_4)_3 \cdot 9\ H_2O$	X	
Ferric chloride	$FeCl_3 \cdot 6\ H_2O$	X	
Cationic polymer	Various	X	X
Calcium hydroxide (Lime)	$Ca(OH)_2$	X*	X
Calcium oxide (Quicklime)	CaO	X*	X
Sodium aluminate	$Na_2Al_2O_4$	X*	X
Bentonite	Clay		X
Calcium carbonate	$CaCO_3$		X
Sodium silicate	Na_2SiO_3		X
Nonionic polymer	Various		X

**Used as a primary coagulant only in water softening processes.*
Source: Belmont Water Treatment Association

6.4.3 Aluminium Sulphate Coagulant

6.4.3.1 Introduction

Aluminium sulphate is a chemical compound with the formula $Al_2(SO_4)_3$. Aluminium sulphate is mainly used as a flocculating agent in the purification of drinking water and wastewater treatment plants, and also in paper manufacturing. It is recommended to be used as the coagulant of choice in Uganda.

6.4.3.2 Storage of Aluminium Sulphate

Aluminium sulphate should be stored in a secured, cool, dry, well-ventilated area, removed from oxidising agents, alkalis, most metals, heat or ignition sources and foodstuffs. Ensure containers are adequately labelled, protected from physical damage and sealed when not in use. Check regularly for leaks or spills (if in a solution form). Large storage areas should have appropriate fire protection and ventilation systems.

6.4.3.3 Health Hazards and Disposal of Waste Solution and Sludge

Aluminium sulphate is categorised as a slightly corrosive, irritant and hazardous substance. This product has the potential to cause adverse health effects with over exposure. Use safe work practices to avoid eye or skin contact and inhalation. It may hydrolyse (with addition of water) to sulphuric acid, a strong tissue irritant.If released to water, aluminium salts will slowly be precipitated as aluminium hydroxide. This may lower the pH of waterways with toxic effects to aquatic organisms. It is not expected to bio-accumulate. Plants may experience chronic toxicity at around 25 ppm. Before disposal, neutralise the solution with lime, weak alkali or similar. For small amounts absorb with sand or similar and dispose of to an approved landfill site.

6.4.3.4 Use of Aluminium Sulphate

Two solution tanks, one for mixing and the other for dosing, between them holding 48 hours of supply, should be provided. The solution strength should be in the range of 5 - 10%. The solution tanks could be equipped with hand agitators as shown in Figure 6-4.

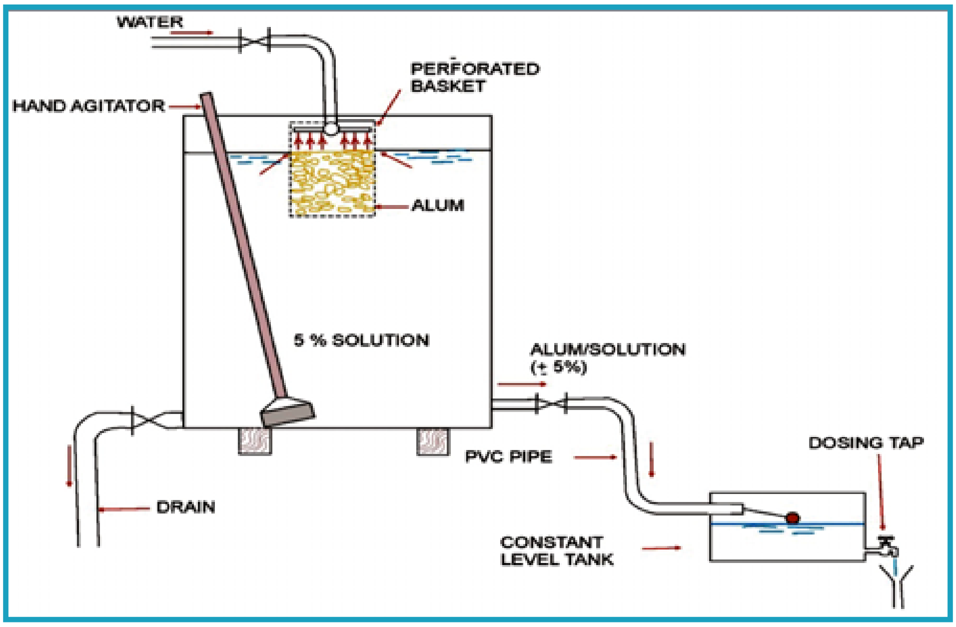


Figure 6-4 Dosing Arrangement for Alum.

If the alkalinity of the raw water is low, the pH can be appropriately adjusted by adding soda ash in the correct proportions, as determined after carrying out laboratory experiments called "jar tests". The strength of the soda ash solution required is usually in the range of 1 - 10%. The solution tanks for soda ash should also hold a total of 48 hours of supply. The chemical solutions should be fed into the raw water by means of gravity dosers, floating balls or other similar simple devices. Dosing pumps should be used only in exceptional cases.

6.4.4 The Jar Test

To determine the correct chemical dosage for aluminium sulphate solution and for water disinfection, jar testing is recommended. Jar testing entails adjusting the amount of treatment chemicals and the sequence in which they are added to samples of raw water held in jars or beakers. The sample is then stirred so that the formation, development, and settlement of floc can be watched just as it would be in the full scale treatment plant. Jar testing should be done seasonally (temperature), monthly, weekly, daily, or whenever a chemical is being changed, or new pumps, rapid mix motors, new floc motors, or new chemical feeders are installed. There is no set requirement for how often jar testing should be conducted, but the more it is done the better the plant will operate. Optimization is the key to running the plant more efficiently.

The jar testing process can be summarized as follows:

i) For each water sample (usually raw water) a number of beakers (jars) are filled with equal amounts of the water sample;
ii) Each beaker of the water sample is treated with a different dose of the chemical;
iii) Other parameters may be altered besides dosage, including chemical types, mixing rate, aeration level/time, filtration type, etc.;
iv) By comparing the final water quality achieved in each beaker, the effect of the different treatment parameters can be determined; and
v) Jar testing is normally carried out on several beakers at a time, with the results from the first test guiding the choice of parameter amounts in the later tests.

6.4.5 Rapid Mixing

Rapid mixing aims at the immediate dispersal of the entire dose of chemicals throughout the mass of the raw water. To achieve this, it is necessary to agitate the water violently and to inject the chemicals in the most turbulent zone, in order to ensure their uniform and rapid dispersal.

Basically, there are two categories of devices used for rapid mixing as follows:

i) Hydraulic rapid mixers; including baffled channels or chambers (see Figure 6-5), overflow (see Figure 6-6) weirs and hydraulic jumps. Gravitational and Hydraulic Types mixers use the kinetic energy of water flowing through the baffles in the plant. A baffled or sinusoidal channel consists of series of baffles around the ends of which the flowing water is reversed in direction, thus causing turbulence or agitation at each point of reversed flow. Turbulence or agitation can also be caused by arranging the baffle so that water flow over and under them. Such channels can either be horizontal or vertical. The baffle system is recommended up to 200 m^3/hr, as no mechanical equipment is needed. This system presents fewer chances of short circuiting and is also simple for construction. The disadvantages with this type of system are more loss of head and space requirement.
ii) Mechanical rapid mixers where the power required for agitating the water is imparted by impellers, propellers or turbines. Mechanical rapid mixers should be used only in exceptional cases. Mechanical mixing should be considered for discharges more than 300 m^3/hr and where the head loss is to be minimized. Whereas for larger plants, multiple units may be provided, for smaller plants, mixing in a tank with paddles is often preferred. The detention period in the mixing device should be between 30 and 120 seconds.

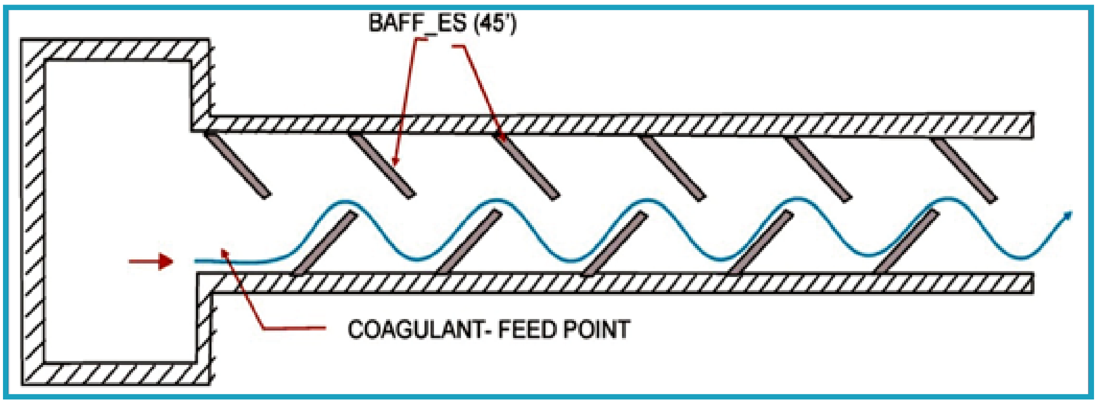


Figure 6-5: Baffled Channel Rapid Mixer

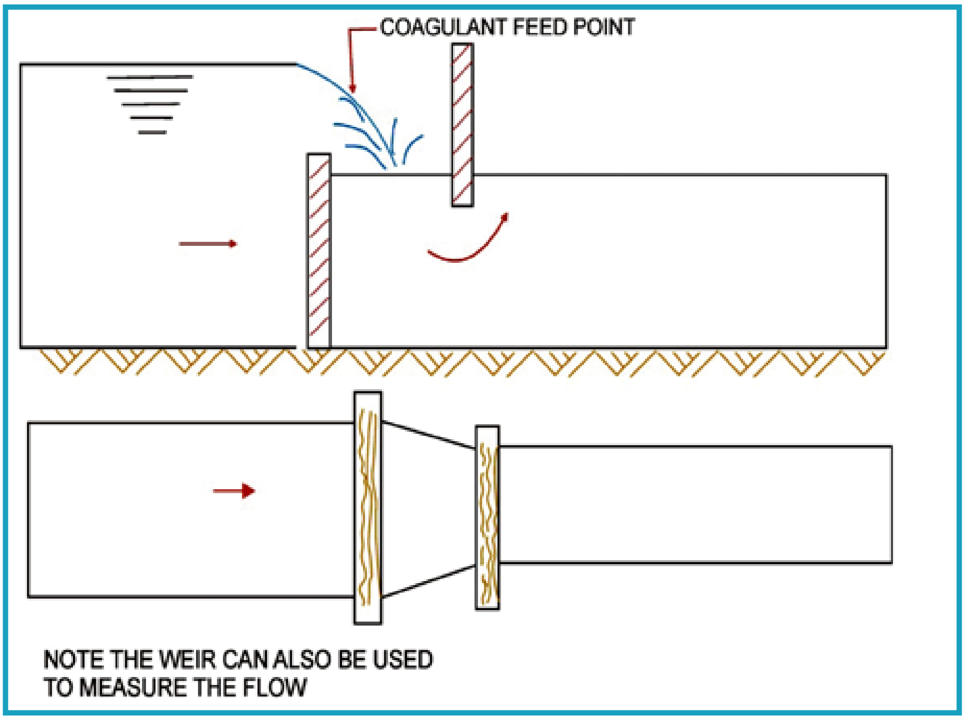


Figure 6-6: Overflow Weir

6.4.6 Flocculation

6.4.6.1 Introduction

Flocculation can be considered to be the stage consequent to coagulation, where the precipitates of finely divided suspended and colloidal matter in water agglomerate to form larger clusters called "flocs" which can then be removed readily by sedimentation and/or filtration. Flocculation is achieved by the process of gentle and continuous stirring of the coagulated water. A number of factors determine the rate of flocculation including particle size, basin and nature of mixing devices, electrolyte concentration, water temperature, pH value, time of flocculation, size of mixing etc.

Basically, there are two categories of flocculators as follows:

i) Hydraulic flocculators including structures such as baffled chambers shown in Figure 6-7 effecting the stirring action. Baffled chambers of the horizontal- flow or vertical-flow types should be designed for a channel flow velocity of 0.1 - 0.3 m/s. The flow velocity in the slots at the inlet end should be 0.6 m/s reducing gradually to 0.3 m/s in the slots at the outlet end; and
ii) Mechanical flocculators where the stirring action is achieved with devices such as paddles. Mechanical flocculators should have a peripheral velocity of 0.9 m/s in the first chamber reducing gradually to 0.2 m/s in subsequent chambers. Mechanical flocculators should be used only in exceptional cases.

Other types of hydraulic flocculators are helicoidal flow, stair case flow, gravel bed and Alabama types. For detailed discussion and design of these flocculators, the manual users are referred to Schulz and Okun, 1984.

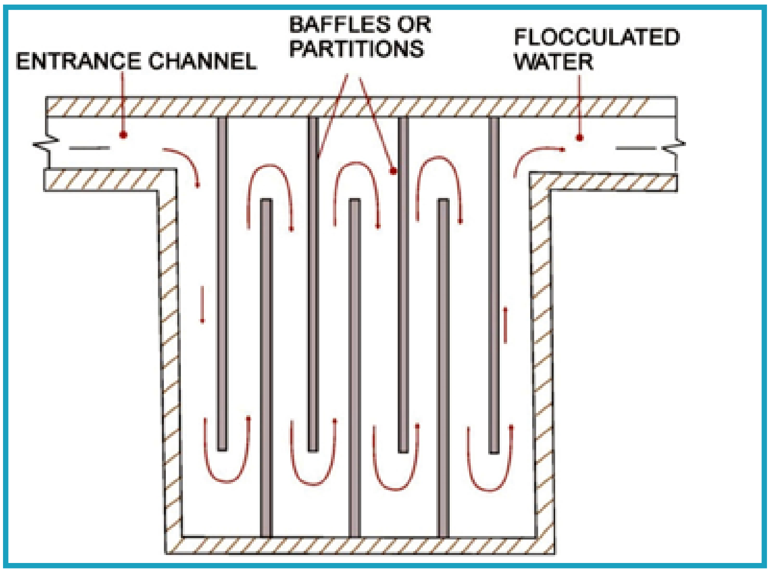


Figure 6-7: Horizontal Flow Baffled Channel Flocculator (Plan)

6.4.6.2 Flocculation Tank Design Criteria

The flocculator design is used to determine the volume of the flocculator tanks and the spacing and number of baffles placed in the flocculator. The computation of volume of the flocculator is as given below:

$$\boldsymbol{V} = \frac{\boldsymbol{P}}{\boldsymbol{G}^2\mu} \qquad \textbf{Equation 6-1}$$

Where:

G	=	Velocity Gradient (s^{-1})
P	=	Power transmitted to the Water (Kw)
V	=	Volume of the flocculation tank (m^3)
µ	=	Kinematic viscosity of water (0.9 x 10^{-6} m^2/s at 25°C)

The flocculator is divided into vertical channels by baffles and the water flows up and down through these channels. The guiding principle is that the length of a channel should be the same as the length of the sedimentation tank. Also, the depth of the water at the end of the flocculator must be the same as the depth of water in the sedimentation tank. Due to head loss through the baffles, the water depth will be greater at the beginning of the tank. Additionally, the design calls for 100mm of free space between the water surface and the top of the tank. Using these assumptions, the baffle spacing is found using the equation below:

$$b = \left(\frac{1}{4}\right)^{\frac{1}{2}} \left(\frac{Q}{w}\right)^{\frac{3}{4}} \left(\frac{K}{2\pi v}\right)^{\frac{1}{4}} \qquad \textbf{Equation 6-2}$$

Where:

Q	=	Flow rate (m3s-1)
w	=	Flocculator width (m)
K	=	Minor loss coefficient (dimensionless)
v	=	Flow velocity (0.9 x 10-6 m2/s at 25°C)

According to Schulz and Okun (1984), the following recommendations can be used for design puposes. Velocity (0.1 - 0.3 m/s), Water depth of at least 1 m, flow rate of 10,000 m^3/day or greater, ideal G values of $15s^{-1}$ - $75s^{-1}$, flocculator tank width of 0.5 - 1m and minor loss coefficient for the 180° bend is 3.

6.4.6.3 Power input and Detention Period

In the design of hydraulic and mechanical flocculators the following factors should be taken into consideration:

i) The velocity gradient G should be in the range of 30 - 60 s^{-1};
ii) The detention time t should be in the range of 900 - 1,200 seconds; and
iii) The product G.t measuring the rate of the floc formation action should be in the range of 30,000 - 100,000.

The power P can be calculated as follows:

$$\mathbf{P = \frac{Qh}{102}} \qquad \textbf{Equation 6-3}$$

Where:

Q = Flow through the flocculation tank (l/s)
h = Head loss in the flocculation tank (m)

It can be assigned that at the velocity head is lost when the water passes through the baffle slot, and that also all the velocity head is lost at all changes of direction of 90° and more. Thus, the losses can be calculated as follows:

$$\mathbf{h = \frac{nv_1^2 + mv_2^2}{2g} + normal\ friction\ losses} \qquad \textbf{Equation 6-4}$$

Where:

h = total head loss, m
n = the number of direction changes
m = the number of baffles
v_1 = the velocity in the channel
v_2 = the velocity in the baffle slot

Total head loss in a flocculation tank usually lies in the range of 150 - 600 mm.

6.4.6.4 Design Criteria

Tapered flocculation chambers are usually made up of two to three compartments in series to minimize short circuiting and the stirrers should be counter-rotating. In order to optimize the velocity gradient, the stirrer in each compartment should be fitted with variable speed motors with that in the last compartment having infinitely variable speed facilities. The compartments should be separated by either round end or over and under baffles arranged to give diagonal flow in the compartment. The head loss across the baffle walls should produce Velocity Gradient (G) value of upto 20 s^{-1}. Flow velocity should be limited to 0.25m/s to minimise floc shear. The tank dimension vary according to the type with horizontal shaft type, the tank should be long and narrow (L:W of atleast 4:1 with a square cross-section perpendicular to the direction of flow) and the depth of about 3m. The tank of the vertical shaft type must be square plan upto 6 x 6m. The depth is not critical provided the stirrer can be supported from the gear box without having to use bottom bearings. The paddle type should be limited to a depth of 4-5m whereas the turbine type could be as deep as 7m and the clearance to the floor should not be less than 500mm. The vertical shaft paddle type stirrer should have a diameter and paddle height greater than 2/3 of the plan dimension and should have clearance to the walls not be less than 500mm. For good performance, the horizontal shaft type should have atleast 3 paddles on each diametrical arm mounted in sinuous channels since short circuiting is reduced. When flocculation tanks are dedicated to individual clarifiers, measures should be taken to include a free fall out to prevent flocculator flow patterns being transmitted to the downstream clarifier and the transfer velocity should be less than 0.1m/s. however, the user is refer to Sincero and Pacquiao (2002) for more detailed design of the individual mixers.

6.4.7 Sedimentation

6.4.7.1 General

Sedimentation is the settling and removal of suspended particles that takes place when water stands still in, or flows slowly through, a sedimentation tank. Sedimentation can be accomplished using horizontal flow tanks, in which the water moves from one end to the other as shown in Figure 6-8, or using vertical flow tanks in which the water enters at the bottom and is taken off at the top surface as shown in Figure 6-9.

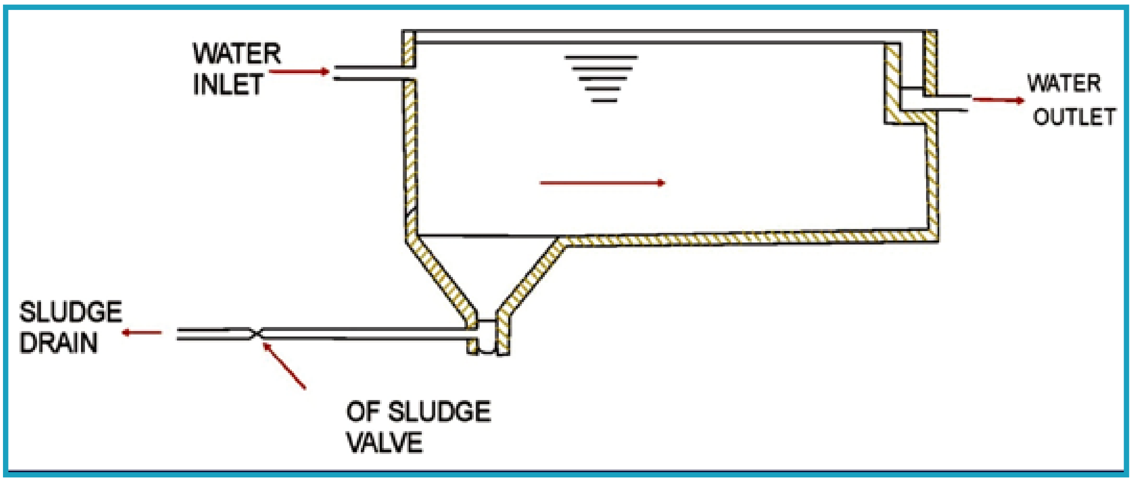


Figure 6-8: Horizontal Flow Sedimentation Tank

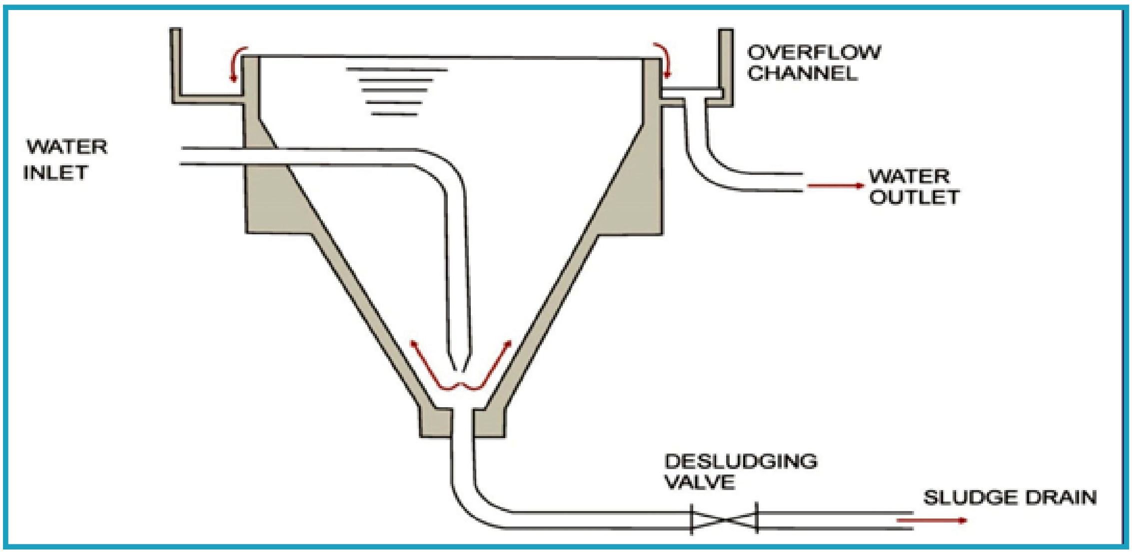


Figure 6-9: Vertical Flow Sedimentation Tank

6.4.7.2 Sedimentation Tank Inlets

Sedimentation tank inlets should be designed in such a way that they distribute the water flow uniformly among the sedimentation tanks and within each individual tank. To achieve this, the inlet openings should be at least 50 mm in diameter, and they should be spaced not more than 0.5 m apart. The inlet flow velocity should not exceed 0.2 m/s. The inlet channel should be sized with a cross-sectional area of at least twice the combined area of the inlet openings. It should be possible to close the inlet to each tank for servicing and cleaning, while the other parallel tanks continue in operation.

6.4.7.3 Sedimentation Tank Outlets

The effluent water should leave the tank over one or more weirs which can be adjusted in level. The total length of the weir (s) in metres should be at least 0.1 x A, where A is the surface area of the sedimentation tank in square metres. In order to keep the residual flocs intact, the outlet velocity from the sedimentation tank should not exceed 0.4 m/s.

6.4.7.4 Horizontal Flow Tanks

To minimize the effects of turbulence, short-circuiting and bottom scour, the ratio of length to width should lie between 3:1 and 6:1 and the effective water depth should be at least 2 m. The tank should be designed with an additional volume of 25% to allow for sludge accumulation. The tank floor should have a gentle slope of 2-3% towards a sludge collection hopper, placed adjacent to the inlet end of the tank. For tanks receiving water that has been treated by chemical coagulation and flocculation, the surface loading should be of the order of 1.0 m^3/m^2h and the detention period 2 hours.

6.4.7.5 Vertical Flow (Sludge Blanket) Tanks

The operational requirements of vertical flow (sludge blanket) tanks are so strict that they are not recommended for use except in exceptional cases. They are to be used only where carefully controlled continuous operation is certain. The surface loading of the tanks should lie in the range of 1.2 to 1.5 m^3/m^2h and the detention time should lie between 1 and 3 hours. The tank walls should be inclined at an angle of 45° - 60° to the horizontal. There should be no obstructions on the walls, apart from carefully positioned sludge pipes.

6.4.8 Rapid Sand Filtration

6.4.8.1 General

Rapid sand filters are used as part of a multiple-stage treatment system. They are widely used and most prefered option of water filtraton in Uganda because of there high capacity water production from smaller land areas compared to slow sand filters. Rapid sand filtration uses relatively coarse sand with an effective grain size in the range of 0.4 to 1.2 mm as the filter medium. The filtration rates are relatively high, generally in the range of 5 to 15 m^3/m^2/hour. Construction details of a typical (open gravity type) rapid sand filter as shown in Figure 6-10.

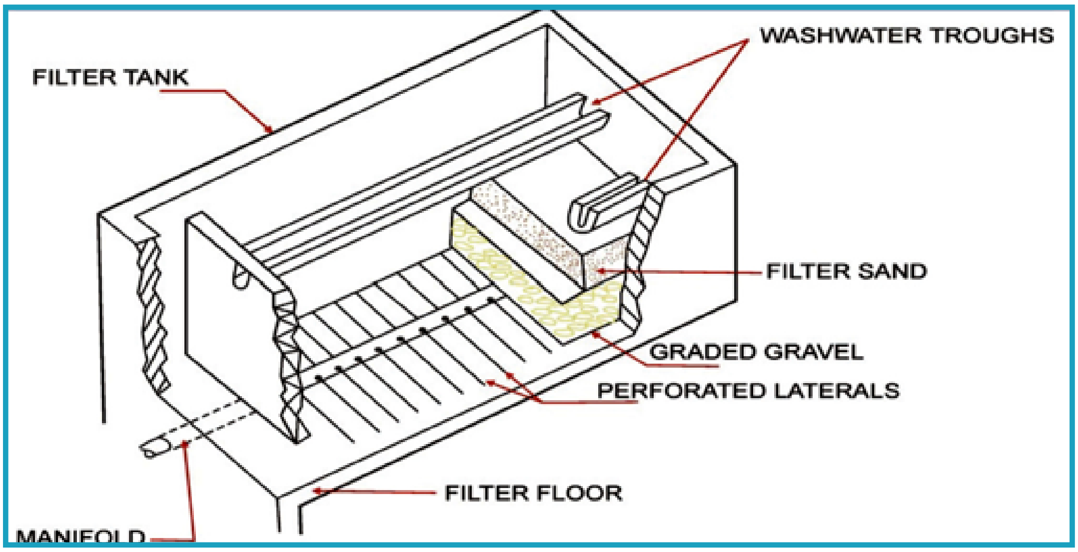


Figure 6-10: Rapid Sand Filter (Open, Gravity Type)

6.4.8.2 Design Details

Each filter unit should have a separate inlet, which can be closed for servicing and backwashing. The inlet should be designed in such a way that flushing and velocities exceeding 0.4 m/s do not occur. The surface loading should be of the order of 5 m^3/m^2h. The height between the top of the filter medium and the bottom of the wash-water troughs should be at least 40% of the height of the filter medium, to allow for the expansion of the medium during backwashing. There should be a special connection to the outlet pipe, through which water filtered during the first 10 minutes after filter backwashing, is discharged to waste.

The following rules of thumb should be applied in the design of the under-drainage system:

i) the ratio total area of lateral pipe orifices: area of fitter bed should be (0.0015 - 0.005):1;
ii) the ratio area of lateral pipe: total area of orifices it serves should be (2 - 4):1;
iii) the ratio area of main manifold pipe: total area of lateral pipes it serves is (1.5- 3):1;
iv) the diameter of lateral pipe orifices should be 5 - 20 mm;
v) the spacing of lateral pipes orifices should be 100 - 300 mm, centre to centre; and
vi) the spacing of lateral pipes should be 100 - 300 mm, centre to centre.

To prevent airbinding of the filter medium, it should be kept submerged by raising the outlet pipe to at least 50 mm above the top level of the medium.

6.4.8.3 Filter Media

The filter medium should consist of a layer of quartz sand of thickness 0.7 - 1.0 m, effective size 0.5 - 1.0 mm and a uniformity coefficient not exceeding 1.5. The medium should be supported by a bed of coarse material (preferably gravel), which will not be dislodged by the backwash water.

The supporting bed can be arranged in four successive layers from top to bottom as follows:

i) 150 mm thick layer grain - sizes 2 - 2.8 mm;
ii) 100 mm thick layer grain - sizes 5.6 - 8.0 mm;
iii) 100 mm thick layer grain - sizes 16 - 23 mm; and
iv) 200 mm thick layer grain - sizes 38 - 54 mm.

6.4.8.4 Backwashing

The filter backwash rates required depend on the grain sizes of the filter media. Typical backwash rates giving about 20% media expansion are given in Table 6-5.

Table 6-5: Typical Backwash Rates (m^3/m^2/hour)

Temperature t (°C)	Filter Media Grain Size d (mm)								
	0.4 mm	0.5 mm	0.6 mm	0.7 mm	0.8 mm	0.9 mm	1.0 mm	1.1 mm	1.2 mm
Backwash Rate v m^3/m^2/hour									
10°C	12	17	22	28	34	40	47	54	62
20°C	14	20	26	33	40	48	56	64	73
30°C	16	23	30	38	47	56	65	75	86

Where:

d = average grain size of filter sand (mm)
t = back-wash water temperature (°C)
v = back-wash rate (m^3/m^2/hour).

To calculate the quantities of water needed for backwashing, it should be assumed that the period for backwashing is 8 minutes. Backwashing with air in combination with water may be used but is not recommended except for big treatment plants operated by skilled personnel. The required backwash water pressure should be determined by calculating the head losses in the water delivery system and in the filter. The total head loss during backwashing from the tank to the backwash water troughs can be of the order of 6 m. The distance which the wash water has to travel horizontally to reach the wash water trough, should not be more than 1.5 m.

Rapid sand filters present much higher flow rates than slow sand filters; require relatively smaller areas of land; are less sensitive to changes in raw water quality and require less quantity of sand. Their operation requires mechanical devices that need electricity for their operation. Rapid sand filters are generally ineffective against taste and odour problems; cannot remove bacteria; produce large volumes of sludge for disposal; require higher operation and maintenance cost and require skilled operators and supervisors compared to slow sand filters.

6.4.9 Slow Sand Filtration

6.4.9.1 General

In slow sand filtration, a bed of fine sand is used through which water slowly percolates downwards as shown in Figure 6-11. Filtration rates are relatively low, in the range of 0.1 and 0.3 m/hour.

6.4.9.2 General Limitations

Slow sand filters expected to perform well if the turbidity of the raw water is less than 10 NTU. Turbidity exceeding 50 NTU is acceptable only for a few weeks, and turbidity exceeding 100 NTU can be tolerated for only a few days. A high content of algae in the raw water may also disrupt the proper functioning of slow sand filters. Some forms of pre-treatment to reduce unacceptably high raw water turbidity and contents of algae will often be necessary. Possible simple pre-treatment methods include prolonged storage, plain sedimentation and roughing filtration as already been discussed.

6.4.9.3 General Design Details

The inlet structure should be designed in such a way that the raw water is equally distributed over the filter bed area. To achieve this, the inlet velocity should be around 0.1 m/s and the width of the inlet structure should be at least (0.05 x Q) metres, where Q is the design flow in m^3/h.

i) The minimum size of a filter unit should be 15 to 20 m^2;
ii) The surface loading should lie between 0.1 and 0.2 m^3/m^2h;
iii) The height of the supernatant water should be 1 to 1.5 m; and
iv) The oxygen content of the water after filtration should not be less than 3 mg/l.

Generally, the following are required for the design of slow sand filter:

i) Rate of filtration or surface loading should be 0.1 - 0.2 $m^3/m^2/hr$;
ii) The minimum size of a filter unit should be 15 to 20 m^2;
iii) The height of the supernatant water should be 1 to 1.5 m;
iv) Determine flow capacity in the filter (water m^3/day-demand); and
v) Calculate filter surface area = flow capacity (m^3) / rate of filtration.

Main Water Under-drain

i) Select flow capacity and flow velocity;
ii) Calculate Diameter of the under drain = $(22 \times d_2) / (28 \times r)$; and
iii) Area of holes or slots to be 1.5% of area of the filter.

As a check, the oxygen content of the water after filtration should not be less than 3 mg/l.

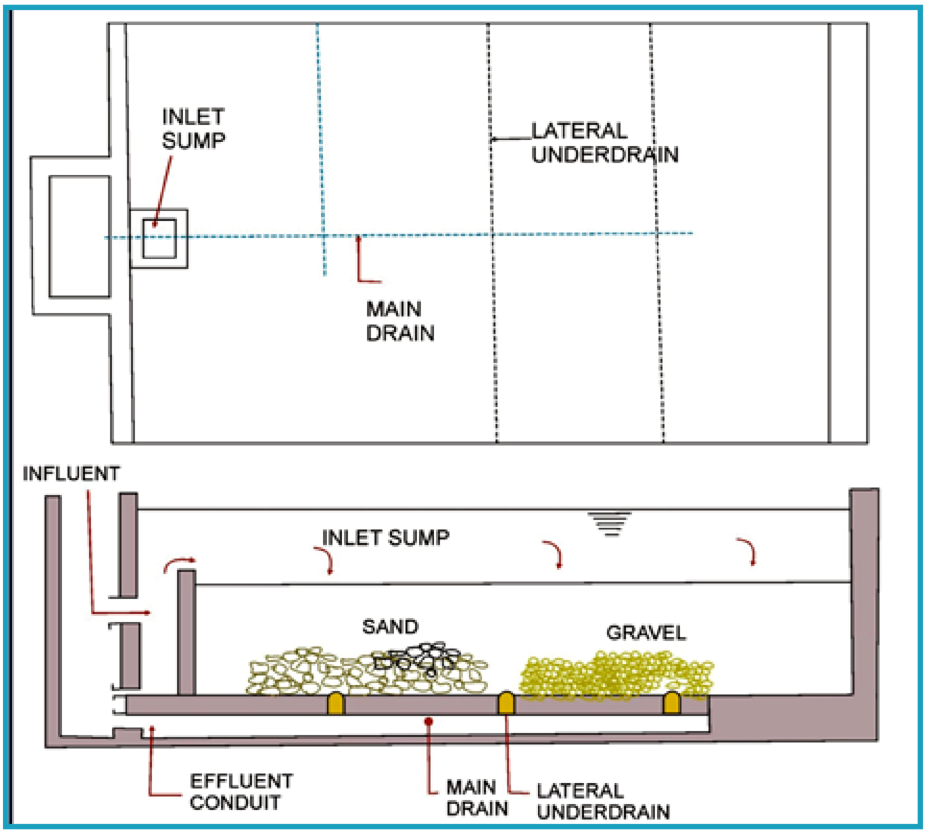


Figure 6-11: Slow Sand Filter

6.4.9.4 Filter Bed

The thickness of the filter sand bed should be 1.0 - 1.4 m. The effective grain size of the sand should preferably lie between 0.15 and 0.35 mm, and the uniformity coefficient between 2 and 5. Ungraded builders' sand, locally available in most parts of Uganda, can serve quite well as filter material. A simple facility such as a platform for washing used filter sand, should be provided as close to the filters as possible.

6.4.10 Sand Filters (SSF) vs. Rapid Sand Filters (RSF)

- **Base material:** In SSF it varies from 3 to 65 mm in size and 30 to 75 cm in depth while in RSF it varies from 3 to 40 mm in size and its depth is slightly more, i.e. about 60 to 90 cm.
- **Filter sand:** In SSF the effective size ranges between 0.2 to 0.4 mm and uniformity coefficient between 1.8 to 2.5 or 3.0. In RSF the effective size ranges between 0.35 to 0.55 and uniformity coefficient between 1.2 to 1.8.
- **Rate of filtration:** In SSF it is small, such as 100 to 200 L/h/sq.m. of filter area while in RSF it is large, such as 3000 to 6000 L/h/sq.m. of filter area.
- **Flexibility:** SSF are not flexible for meeting variation in demand whereas RSF are quite flexible for meeting reasonable variations in demand.
- **Post treatment required:** Almost pure water is obtained from SSF. However, water may be disinfected slightly to make it completely safe. Disinfection is a must after RSF.
- **Method of cleaning:** Scrapping and removing of the top 1.5 to 3 cm thick layer is done to clean SSF. To clean RSF, sand is agitated and backwashed with or without compressed air.
- **Loss of head:** In case of SSF approx. 10 cm is the initial loss, and 0.8 to 1.2m is the final limit when cleaning is required. For RSF 0.3m is the initial loss, and 2.5 to 3.5m is the final limit when cleaning is required.

The major reason why RSF is a preferred option of water filtration to SSF is rate of filtration where RSF is higher per filter area, flexibility where SSF are not flexible for meeting variation in demand, method of cleaning where in the case of RSF, no sand is lost and also RSF requires minimal land requirement.

6.4.11 Pressure Filters

Pressure filters are typically used with hot process softeners to permit high-temperature operation and to prevent heat loss. The use of pressure filters eliminates the need for repumping of filtered water. Pressure filters are similar to gravity filters in that they include filter media, supporting bed, underdrain system, and control device; however, the filter shell has no wash water troughs.

Pressure filters, designed vertically or horizon-tally, have cylindrical steel shells and dished heads. Vertical pressure filters range in diameter from 1 to 3m with capacities as great as 70m^3/hr filtration rates of 0.2m^3/s. Horizontal pressure filters, usually 2m in diameter, are 3-8m long with capacities from 45 to 140m^3/hr. These filters are separated into compartments to allow individual backwashing. Backwash water may be returned to the clarifier or softener for recovery.

Pressure filters are usually operated at a service flow rate of 0.2m^3/s. Dual or multimedia filters are designed for 0.4 – 0.5m^3/s. At ambient temperature, the recommended filter backwash rate is 0.4 – 0.5m^3/s for anthracite and 0.9 -1m^3/s for sand. Anthracite filters associated with hot process softeners require a backwash rate of 0.9 - 1m^3/s because the water is less dense at elevated operating temperatures. Cold water should not be used to backwash a hot process filter (GE power and water, water and process technologies) . This would cause expansion and contraction of the system metallurgy, which would lead to metal fatigue. Also, the oxygen-laden cold water would accelerate corrosion.

6.4.12 Disinfection

6.4.12.1 General

The single most important requirement for drinking water is that it is free from disease-causing (pathogenic) micro-organisms. Processes such as storage, coagulation, flocculation, sedimentation and filtration reduce, to varying degrees, the bacterial content of water. However, these processes are not sufficient to provide bacteriologically safe water. Final disinfection is therefore needed. Disinfection is often the only treatment process required in small water supply schemes, especially those based on groundwater sources and springs. All piped water supply schemes serving several households are to be designed with facilities for disinfecting the water before distribution to the consumers.

It is important that pipelines are designed in such a way as to facilitate disinfection. Disinfection is needed when the pipeline is used for the first time, and later when taking the pipeline back into service after carrying out repairs on it.

6.4.12.2 Chemical Disinfectants

The usage of a particular disinfectant and its dosage should be carefully determined, preferably under laboratory conditions, since overdose of these could easily be harmful to human beings and the environment.

6.4.12.3 Chlorine and Chlorine Compounds

Chlorine and chlorine compounds such as chlorinated lime (bleaching powder), calcium hypochlorite and sodium hypochlorite, are the most effective and commonly used chemical disinfectants. However, these compounds are believed to be carcinogenic with a likelihood of causing cancer. Household disinfectants such as JIK contain the active hypochlorite found in the chlorine solution and could therefore be applied in emergency water disinfection.

Other disinfection methods, such as ultraviolet light and ozone may be used on small treatment plants. The usage of a particular disinfectant and its dosage should be carefully determined, preferably under laboratory conditions, since overdose of these could easily be harmful to human beings and the environment. Table 6-6 shows the different compounds and formulations for chlorine as a disinfectant.

Table 6-6: Chlorine and Chlorine Compounds as Disinfectants.

The name and characteristics of disinfectant	Advantages	Disadvantages
Chlorine Is applied in a gaseous form and requires the strictest safety measures	• Efficient oxidant and disinfectant • Effectively eliminates unpleasant taste and odors • Featured with aftereffects • Prevents and controls growth of algae, biological slimes and microbes • Decomposes organic contaminants (phenols, etc.) • Oxidizes iron and magnesium • Decomposes hydrogen sulfide, cyanides, ammonium and other nitrogen compounds	• Strict requirements for transportation and storage • Potential risk to health in case of leakage • Formation of disinfection by-products, such as trihalomethanes. • Formation of bromates and brom-organic disinfection by-products at presence of bromides
Sodium hypochlorite Is applied in a liquid form (trade concentration - 10 -12%), can be obtained on-site through electrochemical generation.	• Effective against most of pathogenic microorganisms • Relatively safe during storage and use • When produced on site does minimises transportation and storage of hazardous chemicals	• Ineffective against cysts (Giardia, Cryptosporidium) • Loses its activity during long-term storage • Potential danger of gaseous chlorine emission during storage • Produces disinfection by-products, such as trihalomethanes, including bromates and brominated by-products in presence of bromides • Generated on-site requires immediate use, or in case of storage, special measures to purify water and salt heavy metals ions • Generated on-site with concentration of free available chlorine above 450 mg/l and pн >9 accumulates chlorates over time

The name and characteristics of disinfectant	Advantages	Disadvantages
Chlorine dioxide On-site generation only. Commonly excepted as the most effective disinfectant among other chlorine containing agents for water treatment at alkaline pH levels	• Works in small doses • Does not react with ammonia nitrogen • Does not react with oxidizable compounds to form trihalomethanes; destroys some trihalomethane precursors • Destroys phenols that are the source of unpleasant taste and odor • Effective oxidant and disinfectant for all types of microorganisms, including cysts (giardia, cryptosporidium) and viruses • Does not react with bromides to form bromates or brominated by-products • Improves removal of iron and manganese by rapid oxidation and settling of oxidized compounds	• Requires on-site generation equipment • Requires transportation and storage of inflammable chemicals • Forms chlorates and chlorites • In contact with some organic materials and compounds poses unique odor and taste
Chloramine Is formed by mixing of ammonia with free available chlorine	• Commonly used as a disinfectant with prolonged action • Persistent residual • Minimize unpleasant taste and odor • Reduces level of trihalomethanes and haloacetic acid formation • Prevents biofilms formation in distribution systems	• Provides weaker oxidation and disinfection then free chlorine • Is inefficient against viruses and cysts (giardia, cryptosporidium) • Requires increased doses and contact time for disinfection • Presents danger to individuals on dialysis machines, since it can pass through membranes in dialysis machines and induce oxidant damage to erythrocytes • Produces disinfection by-products, including nitrogen-based compounds and chloral hydrate

6.4.12.4 Ultra Violet (UV) radiation

Exposing water to UV radiation effectively inactivates various microorganisms. The most common source of UV radiation is the coated mercury low pressure lamp. To achieve the maximum efficiency, the lamps must be kept clean for the exposure of the organisms to full intensity of the UV light. UV radiation in water disinfection has the following main advantages:

i) It does not require storage and transportation of chemicals;
ii) It does not form disinfection by-products; and

iii) It is effective against cysts (Giardia, Cryptosporidium).

The disadvantages of the UV radiation in the water disinfection include:

i) no residual action;
ii) high maintenance requirements;
iii) high initial capital cost;
iv) high operating (energy) cost;
v) disinfecting activity depends on water turbidity, hardness, bio-fouling of UV lamps, wavelength of UV radiation or power failure; and
vi) does not provides express method for measuring the efficiency of water disinfections.

6.4.12.5 Ozone Disinfection

Ozone can be produced by passing an electric discharge of high voltage alternating current through the dry air. It has been used for several decades for taste and odor control, color removal and disinfection. When ozone is used as a disinfectant, the recommended dosage is 0.2 – 1.5 mg/L.

The main advantages are that:

i) it is a strong disinfectant and oxidant;
ii) it is very effective against viruses;
iii) it is most effective against Giardia, Cryptosporidium, and other known pathogens;
iv) it is capable of enhancing turbidity removal under certain conditions;
v) it is effective in controlling taste and odor; and
vi) it does not form chlorinated by-products.

The main disadvantages of using ozone include:

i) production of disinfection by-products: aldehydes, ketones, carboxylic acids, bromides – containing tri-halomethanes (including bromoform), brominated by-products; brominated acetic acids; peroxides and quinones;
ii) necessity to use biologically active filters to remove by-products;
iii) does not provide residual disinfection effect;
iv) requires high initial expenses for equipment;
v) significant expenses for operators training and installation support; and
vi) reacting with organic compounds, ozone disintegrates them into smaller molecules, which become a feeding media for microorganisms in water distribution systems.

6.4.12.6 Boiling

Other than UV radiation, boiling is another physical disinfection method. It is a safe process which destroys pathogenic micro-organisms. Boiling is effective as a household treatment but not feasible for community water supplies.

6.4.12.7 Disinfection Equipment

Disinfectant solutions should be fed into the water by means of gravity dosers or displacement dosers. Dosage pumps are not recommended except in exceptional cases. Disinfection equipment depends greatly on the type of disinfectant. Gaseous chlorine requires the strictest of controls for both storage and application into the water. It is required that personnel wear or have gas masks on hand. The dosing arrangements are mostly proprietary, with each supplier developing their own systems for dosing and measuring of the output concentrations.

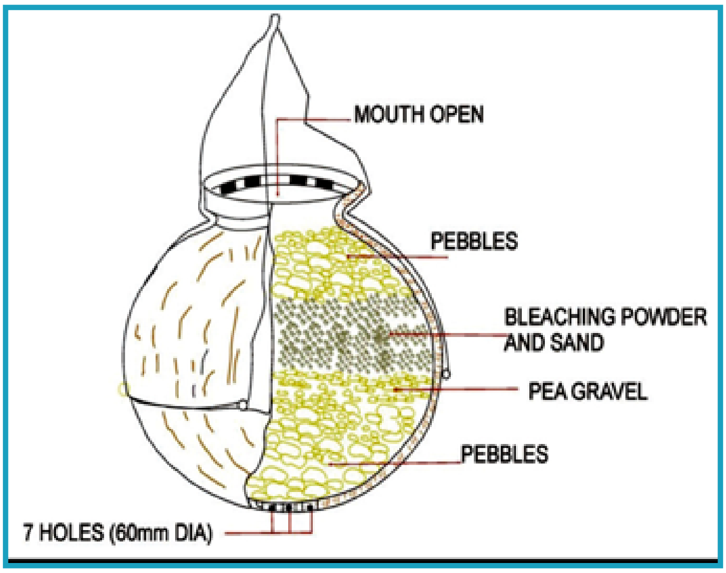


Figure 6 12: Chlorination Pot.

Solid formulations require to be dissolved in a tank from where the concentration is controlled. The supernatant solution is then applied to the water either continuously through a measurement device/ orifice or through batches of water.

The buildings in which chlorine and chlorine compounds are stored and applied to water must be corrosion resistant since chlorine and its compounds are highly corrosive. Safety gear is required in all instances where chlorine and chlorine compounds are to be used for water disinfection. To chlorinate open dug wells in rural areas, a simple pot device like that shown in Figure 6-12 can be used. The pot is lowered into the well with its mouth open.

6.4.12.8 Contact Period and Residual Chlorine

A contact period of at least 30 minutes in the treated water tanks and in transmission distribution mains should be allowed, before the water reaches the first consumer. The residual free chlorine at the consumers' taps should be in the range of 0.3 to 0.5 mg/l.

6.4.12.9 Flow Measurements

Quantities of raw water flowing into the treatment plant can be measured using weirs and flumes which can also serve as the feeding points for the chemical coagulants. There should also be a master (bulk) meter measuring the total quantity of treated water being fed into the distribution system.

6.4.12.10 Water Treatment Using Common Salt

Chlorine can be produced directly from a solution of common salt using electricity, without creating any notable byproducts. This principle has been presented by Grundfos Alldos and is called the selcoperm electrolysis. The reader is encouraged to refer to the Grundfos manual for the application of this technic. The following reactions take place in the electrolytic cell:

$$2NaCl + 2H_2O \longrightarrow 2NaOH + Cl_2 + H_2$$

The chlorine produced reacts immediately with the caustic soda solution also formed, resulting in a hypochlorite solution:

$$Cl_2 + 2NaOH \rightleftharpoons NaCl + NaClO + H_2O$$

The solution generated has a pH value between 8 and 8.5, and a maximum equivalent chlorine concentration of less than 8 g/l. It has a very long half-life which makes it ideal for storage in a buffer tank. After dosing the solution into the water flow, no pH value correction is necessary, as it is often required in electrolysis according to the membrane principle. The sodium hypochlorite solution reacts in a balance reaction, resulting in hypochlorous acid, the efficient disinfectant:

$$NaClO + H_2O \rightleftharpoons NaOH + HClO$$

The dosing quantity depends on the application as well as the local requirement. In general, the concentration after the injection unit is 0.3 to 2 ppm chlorine equivalent.

The benefits of the electrolytic chlorination method

- Common salt is the base material – it is nontoxic and easy to store.
- Only water, common salt and electricity is needed for the electrolysis – low operating costs.
- Fresh hypochlorite is always on hand – the disinfectant solution does not dissociate like commercial hypochlorite solutions. Lower formation of chlorate as a by-product.
- Approved disinfection method complying with drinking water regulations – an alternative with less safety requirements to chlorine-gas-based systems.
- Robust and elementary components – low-maintenance and a long service life, compared with the membrane cell electrolysis

The systems are a good alternative to chlorine-gas-based systems. Key components in the Selcoperm unit include an electrolysis system, storage tanks for the saturated salt solution and product solution, an exhaust fan for removing any hydrogen produced, and dosing pumps for the product solution. The concept is rounded off with a measuring and control unit for chlorine dosing. The electrolysis system is supplied as a turn-key solution. Only the pipework for the water supply, the connections of the salt and product storage tanks as well as the exhaust pipe have to be installed. The size of the storage tanks depends on the capacity level of the system and the amount of disinfectant solution required.

6.4.13 Water Treatment Plant Buildings

6.4.13.1 Workshops, Laboratories and Offices

A workshop with working benches, shelves, lockable cabinets and tools should be provided. A laboratory equipped with sinks, benches, shelves, lockable cabinets and a laboratory kit should also be provided in a separate room. The laboratory kit provided should be adequate to cater for the experiments that are to be done in the laboratory. For small water supplies, some of the experiments may be carried out from a larger nearby laboratory.

Workshops and laboratories should be provided with adequately designed floor drains. Offices should be placed some distance away from noisy equipment such as pumps and generators, preferably in a separate administration building. Pump houses should be designed considering that the heavy pumps and control panels may have to be moved using gantry cranes mounted on steel girders.

6.4.13.2 Stores

All water supply schemes should be provided with adequately sized and secure stores for the storage of chemicals, fuels, pipes and other bulk materials. Stores for corrosive materials and chemicals should be able to resist the corrosion. Explosive and poisonous chemicals such as chlorine should be adequately secured to ensure that accidents are minimised and if they occur, escape routes are clearly marked. All personnel should be trained on the safety procedures. Safety signage must be prominently displayed throughout the plant.

6.4.13.3 Staff Houses

The required numbers and types of operation staff houses should be determined in consultation with the Ministry of Water and Environment, Directorate of Water and Environment.

The houses should be placed some distance away and fenced off from treatment plant, pumping stations and other publicly used facilities. As much as possible, staff housing should be located outside the micro-catchment which is the source of the raw water. Septic tanks, pit latrines and waste dumps should be located such that they do not pollute the raw water source.

6.5 Miscellaneous

6.5.1 Access Roads

Adequately designed access roads should be provided to all buddings such as offices, stores and pump houses, etc.

6.5.2 Safety Measures and Devices

Safety measures and devices such as lighting, fencing, hand rails, fire extinguishers and first aid kits are to be provided wherever needed. The strategic importance of water treatment facilities must be borne in the designer's mind; all efforts to ensure that the water treatment plant is secured and can be protected from intrusion, accidents, terrorism and spillage of pollutants should be expended. If these factors are considered at the design stage, it will be easier and cheaper to safeguard the facility.

Chapter 7

WATER TRANSMISSION AND DISTRIBUTION

7.1 General Aspects

The purpose of a transmission and distribution system of a water supply scheme is to deliver the right quantity and quality of water conveniently to the demand areas. The delivery system is classified into two: the transmission main and the distribution. Transmission mains convey water from the source, treatment, or storage facilities to the distribution system normally via a storage reservoir. There maybe a few service connections on the transmission main, but the purpose of this larger diameter pipe is to deliver water to the distribution mains where most of the service connections are. Distribution mains deliver water to individual customer service lines and provide water for fire protection through fire hydrants, if applicable. The distribution mains normally deliver water from a storage reservoir to the consumers.

The extend and routing of the transmission and distribution network depends largely on the location and characteristics of the demand areas to be served, the topography of the service area and the source of water, the quantities of water to be transmitted and the available infrastructure in the area such as roads, underground facilities and infrastructures etc.

7.2 System Design

7.2.1 Introduction

A given supply area may be best served by one large scheme or by several independent smaller schemes. Alternative systems have to be studied and compared from technical as well as economic viewpoints. The economic analyses should include both the local and foreign exchange components of the capital and operation and maintenance costs over the schemes design period. In this regard, a gravity supply system with high capital costs may be preferred to a pumped system with lower capital costs but with higher operation and maintenance costs in the long term.

Within a given supply area, sparsely populated peripheral and isolated high areas should be studied carefully, to determine whether they could be more economically served by separate systems (springs, wells, booster pumps, rain harvesting etc.) rather than by the main supply system. The costs of the transmission and distribution pipelines per cubic metre of water supplied will generally increase with the size of the supply area. However, this increase will often be off-set by the decreased costs for water treatment and pumping for the larger supply area.

The most economical supply area should be determined uniquely for each project, as it is not possible to give general rules concerning optimal scheme sizes. However, simple gravity schemes without treatment should generally be smaller and more preferable than complicated schemes with treatment and pumping. In most cases, the construction of large schemes will have to be done in phases. This factor should be considered right from the preliminary design stages, so that the schemes can be appropriately divided into technically and economically manageable schemes.

7.2.2 Pipeline Design

The purpose of pipeline design is to properly control frictional energy losses so as to move the desired flows through the system, by conserving energy at some points and burning it off at other points. This can be accomplished by careful selection of pipe sizes and strategic location of control valves, break pressure tanks (BPT), reservoirs, tap stands, etc. Designing pipelines requires the basic hydraulic principles that govern the behaviour of water flowing in a closed conduit. The general procedure for designing a pipe network is as follows:

i) Identification of demand points (consumer locations) and plotting them on a sketch map
ii) Drawing a sketch of the pipeline connecting the located consumer points putting into consideration high points (crests) and low points (valleys); available infrastructure such as roads, properties etc. and future development plans of the area.
iii) Determine the average and peak water demands estimated from the populations and water use at each of the supply points
iv) Carry out a topographic and cadastral survey of the supply area putting particular emphasis on the identified pipeline route and consumer points. Change the route if necessary if topography dictates so.
v) Establishing a vertical profile in each of the pipe sections connecting important features such as junctions, tap connections
vi) Using the peak water demands at each supply point and the ground levels carry out a hydraulic design for each of the pipe sections so as to determine the pipe diameter, flow rate, velocity of flow, static pressure and hydraulic pressure in the pipe sections. This can be done using computer software.

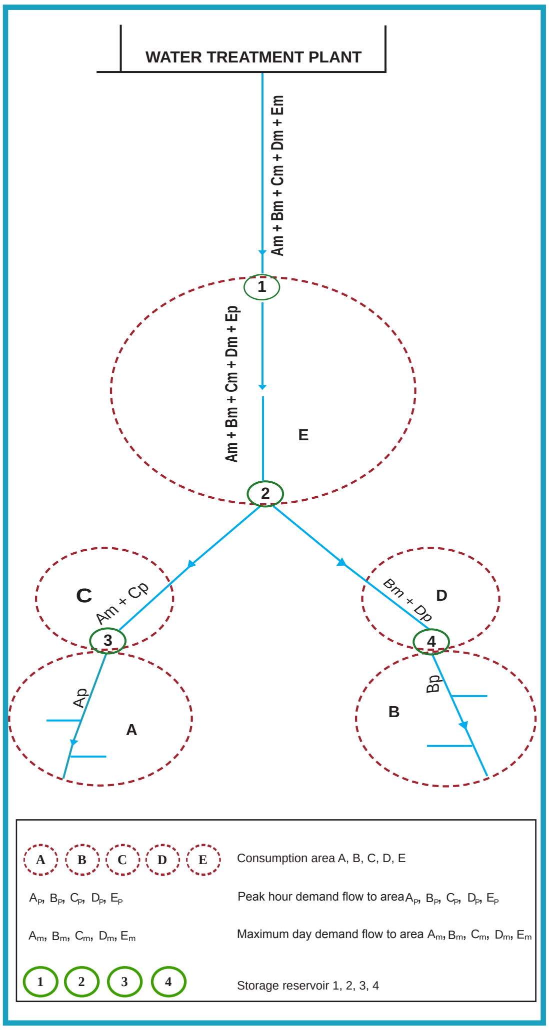


Figure 7-1 Water Flow Transmission

The hydraulic design ends when all sections of the pipeline network have been allocated pipe diameters, velocity of flow, static pressure, hydraulic pressure and head loses. Once the profile has been plotted and the final pipeline sizing completed, the design of other components of the water supply system such as the intake, reservoir tank, tap stands, air valves, washout valves etc. follows.

The costs of the transmission and distribution systems constitute the bulk (80 – 90%) of the overall costs of water supply schemes. It is therefore important that the design of these systems is optimized through careful choice of pipe materials and alignment of the pipeline routes.

7.2.3 Design Requirements for Branch Systems

The design of distribution system in a branch or dead end system is very simple. The procedure is as follows:-

i) Detailed maps are prepared for each area showing the streets and the blocks of houses, parks etc.
ii) A tentative layout of the distribution pipes is then drawn along the streets and the distribution of flow is marked by arrows.
iii) The position of sluice valves, fire hydrants etc. are marked.
iv) The population to be served by each section of the pipe length between two sluice valves and between junctions or branching off points is noted against each section.
v) Knowing the unit rates of demand, the quantity of water to be supplied in each section can be determined.

vi) Then starting from the farthest end and proceeding towards the distribution reservoir or the pump house, the total quantity to be carried by each section of the pipe length can be summed on a cumulative basis.

vii) This gives the average flow the pipes have to carry whilst the maximum flow will be 2.5 to 3.0 times the average. This depends on the supply position also. If the town is supplied intermittently only, the maximum flow rate would be 4 to 5 times the average rate. To this flow, the fire demand and other special requirements should then be added.

viii) The pipe diameters are initially calculated assuming that velocities of flow generated in the pipe lie between 0.6 to 2.00 m/s and that they are lower in smaller diameter pipes and higher in the larger pipes. The recommended velocities are given in Table 7-1

Table 7-1 Guide to Distribution Velocity Vs Diameter

Pipe Diameter Range (mm)	Velocity Range (m/s)	Nom. Dia. (mm)	Velocity Range (l/s)
Small, DN 50 - 110	0.60 - 1.00	50 65 80 100 110	1 – 2 2 – 3 3 – 5 5 – 8 6 - 10
Medium, DN 125 - 250	1.00 - 1.50	125 150 200 225 250	10 – 18 18 – 30 30 – 45 40 – 60 50 - 80
Large, DN 300 - 500	1.20 - 2.00	300 350 400 450 500	80 – 140 120 – 190 150 – 250 190 – 300 250 - 400

Because internal diameters differ depending on pipe, these values should be used only as an initial guide, the selection to be reviewed once the pipe material is known.For larger pipe diameters the manufacturers recommended values should be taken.Based on the flow and velocity criteria, the tentative pipe sizes are selected. Thus the pipes are selected for the first round trial.Once the diameter of the pipes, the velocity of flow and the length are known, the head lost due to friction in the pipe can be found by a suitable friction loss formula. Because applying thisformula and getting the friction loss in every section of a large distribution system is very tedious and cumbersome, discharge tables showing friction losses can be used or preferably a computer spreadsheet. If the latter is used then the preferred formula is that of Colebrook-White as a simple iterative procedure built into the spreadsheet will perform the calculation.

Thus after getting the head lost due to friction in various pipe lengths, the residual pressures remaining throughout the network can be calculated by deducting these from the initial pressure head available at the distribution reservoir. These should be sufficient for the water to reach the highest and/or farthest point of supply with the minimum residual pressure. If this is not achieved then a next larger diameter in any salient section should be assumed. If the residual pressure is very high than pipe diameters in those and preceding sections should be decreased so that a combination is achieved for an economical system. Several trials in computation may be necessary to achieve this.

A final computation is then required once the detailed design of the system is nearing completion, and particularly where thermoplastic pipes are included, the final wall thickness have been determined.

7.2.4 Economic Pipe Sizing

Economic pipe sizing aims at supplying the maximum number of consumers at the lowest possible cost. In practice, this can be achieved by observing the following rules:

i) Storage (balancing) reservoirs should be incorporated in the systems, in order to cut down on pipelines designed for peak hour flows. The positions and capacities of the reservoirs should be determined after performing appropriate analyses aimed at minimizing system costs;
ii) A pipeline traversing a given supply area, should be designed with adequate capacity to carry the Peak Hour Demand required at the supply area plus the "Maximum Day demand". Determination of demand is covered in Chapter 2 on Water Demand;
iii) Static pressures should be kept as low as possible, by breaking the pressure preferably in the storage (balancing) reservoirs or in special break-pressure tanks. Pressure reducing valves are not recommended except in exceptional cases;
iv) To avoid air pockets, the numbers of pronounced high and low points should be kept to the minimum, by trying to follow the contour lines of the terrain rather than following roads and tracks. This calls for the active participation of the designer in the survey of the pipeline routes; and
v) In order to minimize the number of air-valves and washouts, pipeline excavation depths should be varied to avoid local high and low points.

In general, the most economic pipe diameter will be large in the following circumstances:

i) When energy costs are relatively high; and
ii) When costs per linear meter of pipeline are low when interest rates on capital are low.

The costs of transmission and distribution systems are more sensitive to the total lengths of the pipelines installed than to the sizes of the pipelines. Therefore, it is advantageous to design systems (or the major components of the systems) at once to meet the projected "Ultimate" Year demands.

In the design of water distribution system, a number of software may be used including EPANET, WaterCAD etc. One of the most commonly used and freely available software for the design of water distribution system is EPANET. EPANET software can be obtainable online from http://www.epanet.de/ [accessed Wednesday, 11th April, 2012].

EPANET is a computerized simulation model designed to meet the need to understand movement and transformations that treated water undergo in a distribution system. The model predicts the hydraulic and water quality behaviour over an extended period of time. Figure 7-2 shows the physical components in water distribution system.

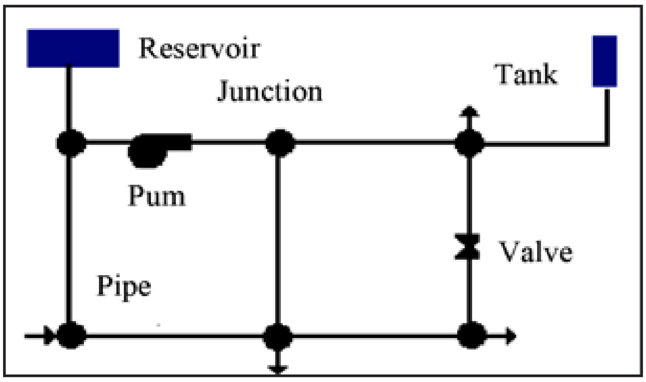


Figure 7-2: Physical Components in Water Distribution System
Source: EPANET User's Manual, 2000

The following steps are followed when using EPANET to model a water distribution system:

i) First and foremost, the network representation of the distribution system is drawn; alternatively, the basic description of the network placed in a text file can also be imported. EPANET uses various types of objects which can be accessed either directly on the network map or from the data page of the browser window to model a distribution system. The objects are classified as: Nodes (Junctions, Reservoirs, and tanks), Links (pipes, pumps, Valves), Map labels, Time patterns, Curves and Controls (Simple or Rule based controls);
ii) Using the property editor, the properties of the objects that make up the system are edited. This involves editing the properties of the visual (pipes, pumps, valves, tanks). To edit an object, the object on the map is selected then 'Edit' on the data browser is clicked;
iii) The next step is to describe how the system is operated. The non – visual objects (curves, time patterns and controls) are edited using special editors that control their properties;
iv) Analysis options are then selected. After suitably describing the network, its hydraulic and water quality behaviour can be fully analysed. The five options that control how EPANET analyzes a network are Hydraulics, Quality, Times, Reactions and Energy. These options are selected from the data browser or the menu bar from the project menu;
v) A water quality/hydraulic analysis is run by selecting the project then running analysis on the standard tool bar. The analysis is ended by clicking OK; and
vi) The results of the analysis can be viewed on the network map as graphs or in tables or as special reports.

7.2.5 Hydraulic Calculations

7.2.5.1 Introduction

Once the flow rate to be conveyed by each pipe section, the length of the pipe and the ground levels of the pipe sections are determined, the hydraulic parameters of the pipe section can be calculated by applying hydraulic formulae. Hydraulic calculations are carried out by application of the pipe friction head loss equations.

7.2.5.2 Hardy Cross Method

The Hardy Cross method is an iterative method for determining the flow in a pipe network system where the inputs and outputs are known, but the flow inside the network is unknown. The method was published by Hardy Cross in November 1936 and was an adaptation of the Moment distribution method. The application has been replaced by complex computer solving algorithms like Newton-Raphson and others but can be used to check the results from such methods. The Hardy Cross method applies continuity of flow and potential to iteratively solve for flows in a pipe network using Hazen - Williams or Darcy - Weisbach equations. The general formulae provided by Hazen – Willam from which head loss can be computed is;

$$V = kCR^{0.63}S^{0.54} \qquad \textbf{Equation 7-1}$$

Where V is pipe flow velocity, k is a conversion factor (0.849 for SI units), C roughness coefficient R hydraulic radius and S the slope of the energy line.

$$S = \frac{h_f}{L} \qquad \textbf{Equation 7-2}$$

Where h_f is head loss and L is pipe length. Also from continuity of flow equation

$$Q = VA \text{ where } A = \frac{\pi d^2}{4}$$ **Equation 7-3**

Where Q is pipe flow, A is the pipe cross section area and d the pipe diameter

The complete derivation of hf from Hazen-William equation with C values are given in Appendix 6. However, for simplified calculation of head loss, the equation below can be used, where n is taken as 2.

$$h_f = KLQ^n$$ **Equation 7-4**

In Hardy Cross balance of heads method, continuity of flow at each node is iteratively satisfied given initial flow conditions and then the flows are balanced until continuity of potential is also achieved over each loop in the system.

For a given network, an initial pipe flow value Q_o is assumed such that the law of continuity is satisfied at each junction. The initial flow value selected will determine the number of iterations to be made. For each loop, the head losses h_f are determined clockwise and counter-clockwise and the summation should be zero.

The assumed flows should then be corrected using ΔQ as shown below and added to the initial guess until there is no significant change.

$$\Delta Q = -\frac{\sum h_f}{n \sum {h_f}/{Q}}$$ **Equation 7-5**

7.3 Walking Distance to Water Points

7.3.1 Rural Areas

Generally in the rural areas, the distance from 90% of the households to the nearest primary or secondary pipeline should not exceed 1.5 km. However, local conditions such as the terrain, the spacing of settlements and tracks and others may necessitate adjustments to this guideline, in order to serve the maximum number of people in the most effective way.

7.3.2 Urban Areas

In urban areas, pipelines should follow roads and streets as much as is possible. Presence of other underground infrastructure such as telephone cables, electricity cables, fibre optic system and sewers should always be anticipated and catered for in the earlier stages of selecting the pipe route.

7.4 Pipes

7.4.1 Materials Selection

Pipe materials commonly used in Uganda include ductile iron (DI), steel, galvanized steel (GS), asbestos cement (AC), un-plasticised polyvinyl chloride (uPVC) and polyethylene (PE).

The suitability of a given pipe type for a particular application is influenced by the following factors:

i) Its availability on the market in respect of sizes and pressure classes;
ii) Its cost price and that of its associated valves and fittings;
iii) Susceptibility to corrosion, mechanical damage, ageing and other causes of material deterioration; and
iv) Storage costs.

Corrosion aspects should be considered carefully in the process of pipe material selection. For steel and iron pipes, external corrosion can pose a much greater problem than internal corrosion. The same constituents that affect pipes from the inside can also attack from the outside. Soil resistivity and corrosion have a close relationship and soil resistivity values of less than 7 Ohm-m indicate highly corrosive soil conditions. In such conditions, special protective measures such as cement mortar or coal-tar linings and coatings, or cathodic protection should be provided as recommended by the pipe manufacturers.

DI, GS and steel pipes are strong, and they are the pipes of choice for very high operating pressures and for large size pipelines exceeding 300 mm in diameter. However, the costs of fittings and valves increase rapidly with higher pipe pressure classes. It is therefore necessary to keep reducing pipe pressures through the provision of break-pressure tanks or pressure release valves wherever appropriate. These pipes have a further advantage that unauthorized tapping from DI, GS and steel pipelines is not easy.

Owing to the health hazards associated with the inhalation of asbestos fibres during the manufacture and installation of AC pipes they are no longer used for domestic water supplies. uPVC pipes have the advantage of easy handling and installation and also high corrosion resistance. However, the pipes suffer some loss in strength when exposed to direct sunlight, and care should therefore be taken to keep them sheltered when they are in the open. uPVC pipes are also easily damaged by careless handling and transportation. The pipes are manufactured in several classes to satisfy different design and operational pressures and they are generally most suitable for use in small size pipelines of diameters not exceeding 160 mm. These pipes have a disadvantage that unauthorized tapping from UPVC pipelines is quite easy.

Polyethylene is very suitable for small-diameter pipelines because it can be supplied in rolls, thus reducing the numbers of the joints and bends required. PE pipes are supplied as High Density (HDPE) or medium density (MDPE). HDPE is preferable although it is supplied at a higher cost. Polyethylene does not deteriorate when exposed to direct sunlight. However, they are not suitable in rocky ground as they can be susceptible to damage. The pressure ratings for PE pipes should be adhered to strictly as they show a high rate of pipe bursts. These pipes like uPVC have a disadvantage that unauthorized tapping from polyethylene pipelines is quite easy.

7.4.2 Pipe Sizes

Table 7-2 Pipe Sizes Available in Uganda

Pipe Types	Sizes (mm)
uPVC	63, 90, 110, 160, 200, 250, 315, 400
PE	20, 25, 32, 40, 50, 63, 75, 90, 110, 125, 140
GS and steel	15, 20, 25, 32, 40, 50, 65, 80, 100, 150, 200, 250, 300

Table 7-3: Polyethylene Pipes Commonly Used in Uganda

Outside Diameter	Pressure Rating				
(mm)	PN 2.5	PN 4	PN 6	PN 10	PN 16
	Inside Diameter (mm)				
20				16.0	14.4
25			21.0	20.4	18.0
32			28.0	26.2	23.2
40		36.0	35.4	32.6	29.0
50		46.0	44.2	40.8	36.2
63	59.8	58.2	55.8	51.4	45.8
75	71.2	69.2	66.4	61.4	54.4
90	85.6	83	79.8	73.6	65.4
110	104.6	101.6	97.4	90.0	79.8
125	118.8	115.4	110.8	102.2	90.8
140	133.0	129.2	124	114.6	101.6
160	152.0	147.6	141.8	130.8	116.2

7.4.3 Pipe Friction Losses

The procedure of determination of pipe friction loses is what culminates into hydraulic design of a pipe. This procedure should be carried out in every section of the pipeline. The outcome of this procedure is the selection of pipe diameter that meets the head lose, pressure and flow design criteria. Pipe manufacturers usually provide pipe friction loss data either in tables or, more commonly, in form of charts (nomographs). The data provided is for new pipes and it is prepared with no consideration for the following:

- Changed internal pipe wall roughness over time
- Additional head losses caused by the valves and fittings installed in the pipeline.

Pipe diameter and the hydraulic pressure in a pipe are derived from the friction losses in the pipe. To calculate friction losses in ordinary transmission and distribution pipelines, it is recommended that the following empirical formulae be used:

1. Colebrook – White (Universal) formula;
2. Darcy – Weisbach formula; or
3. Hazen – Williams formula.

Friction losses in individual valves and fittings can be calculated separately from charts given in Chapter 8 – “Water Pumping”.

7.4.4 Pipe Cover and Slopes

Pipelines should be laid in straight lines between changes in gradient, and pipeline slopes should at no place be less than 0.5%. Local high points where air pockets can develop in a pipeline, without having the chance of being released, must be avoided. To minimize changes in pipeline slopes, pipe cover can be varied from a minimum of 0.6 m to a maximum of 3 m. However, the minimum cover over pipelines laid below road surfaces and reserves should be 0.9 m. Such pipelines should be laid and protected in the manner required by the Ministry responsible for road works.

7.4.5 Pumping Mains

Water hammer/ surge must be taken into consideration when designing pumping mains. For uPVC pipes, the total pressure variations from minimum to maximum should not exceed 50% of the nominal working pressure of that pipe class. The most economical pipe diameter for a pumping main should be determined through appropriate economic analyses. For preliminary estimates however, the most economical size of a long pumping main can be found using a velocity of flow of 0.8 m/s in the main. The design of short suction and discharge pipe work installations in pumping stations should be done as explained in Chapter 8 – “Water Pumping”.

7.4.6 Pipeline Pressures

In general, the minimum pressure in the sections of a pipeline where consumer service connections will be made should be 100 Pa (10 meters of water head). However, in determining this minimum pressure, the elevations of the surrounding areas to be served from the pipeline must also be taken into consideration. The static pressure in pipelines having consumer service connections should not be more than 600 Pa (60 meters of water head) unless the terrain makes higher pressures unavoidable. Whenever the static pressure in the pipeline section gets to 100 m provision for breaking the pressures such as break pressure tanks should be made. Service pressures higher than 600 Pa may necessitate the provision of special fittings, bail valves and stop valves in the consumer service connection lines.

7.4.7 Residual Pressure

In designing the distribution system, minimum pressure at a public kiosk or yard tap should be atleast 10meters but not more than 25 metres. Public supply points fed from pumping mains should be kept to a minimum with small balancing tanks provided off the pumping main wherever practicable from which several adjoining public taps are then supplied. In undulating terrain, upto 150 metres pressure may be allowed. For storied building owners or occupiers,individual arrangements for boosting the water where the pressure is not sufficient should be made. The pressure required for fire fighting at the hydrant should not be less than 15 metres and should not be more than 60 metres in the mains.

7.4.8 Water Hammer / Surge

Water hammer (surge) is a phenomenon which occurs frequently in pressure pipelines, and can be defined as the periodic pressure oscillations which move back and forth along a pipeline. The phenomenon can be caused by closing or opening a valve, starting or stopping of a pump or when any other operating conditions in the pipeline change. In such circumstances, the water column in the pipeline may not adapt to the new conditions immediately, and during the interim period, different parts of the water column will collide with each other violently. Water hammer is usually recognized by a banging or thumping in water lines.

The causes of water hammer are varied. There are, however, four common events that typically induce large changes in pressure:

i) Pump start-up can induce the rapid collapse of a void space that exists downstream from a starting pump. This generates high pressures;
ii) Pump power failure can create a rapid change in flow, which causes a pressure upsurge on the suction side and a pressure down-surge on the discharge side. The down surge is usually the major problem. The pressure on the discharge side reaches vapour pressure, resulting in vapour column separation;
iii) Valve opening and closing is fundamental to safe pipeline operation. Closing a valve at the downstream end of a pipeline creates a pressure wave that moves toward the reservoir. Closing a valve in less time than it takes for the pressure surge to travel to the end of the pipeline and back is called “sudden valve closure”. Sudden valve closure will change velocity quickly and can

result in a pressure surge. The pressure surge resulting from a sudden valve opening is usually not as excessive; and

iv) Improper operation or incorporation of surge protection devices can do more harm than good. An example is over sizing the surge relief valve or improperly selecting the vacuum breaker-air relief valve. Another example is to try to incorporate some means of preventing water hammer when it may not be a problem.

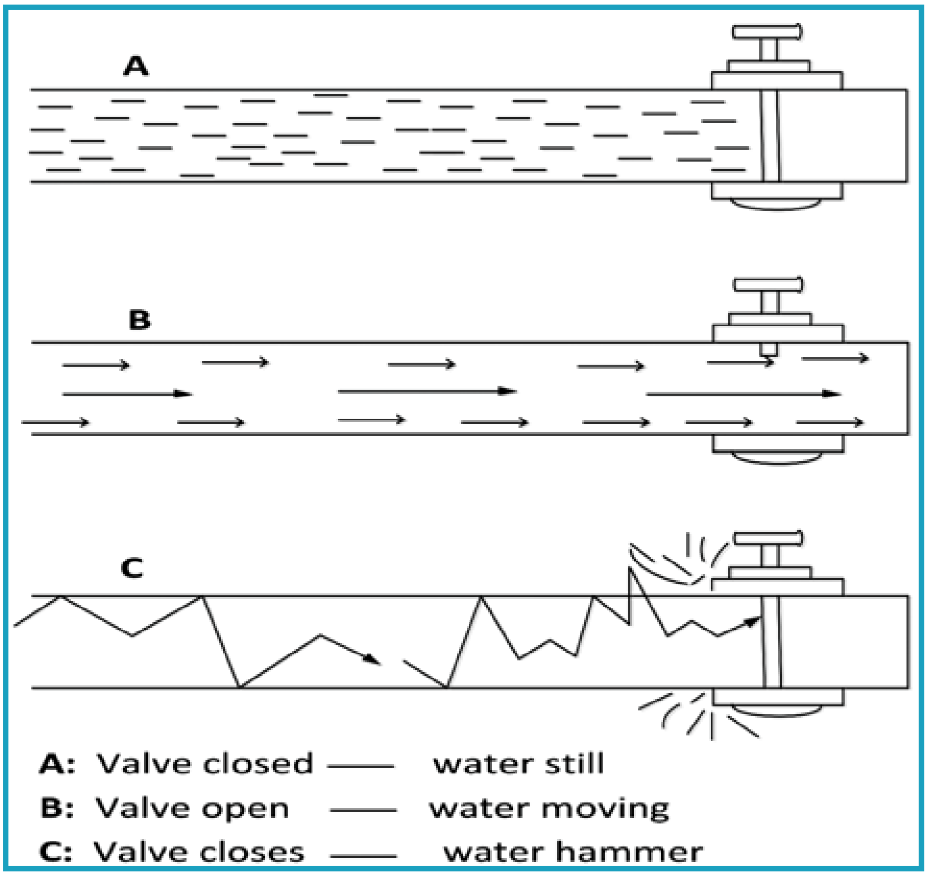


Figure 7-3: Water Hammer Concept

The crucial factors in determining water hammer (surge) pressures are as follows:

i) pipeline length;
ii) profile of the pipeline route; and
iii) water flow velocity in the pipeline.

Water hammer pressures are often the most critical factor in the design of pressure pipelines. The risk of damage to pipelines by water hammer pressures makes it necessary to install surge tanks or accumulator tanks or to follow special operating procedures to minimize this risk. However, in many cases the most economical solution may be to use pipes of higher pressure classes than would normally be required.

The elasticity moduli and Poisson’s ratios of different types of pipe are given in Table 7-3. The values of the factor $ for uPVC and steel pipes are shown in Table 7-4.

Table 7-4 Elasticity Moduli and Poisson's Ratios

Material	Elasticity Modulus E_p N/mm²	Poisson's ratio
Un-plasticised Polyvinyl Chloride (uPVC)	3×10^3	0.5
Polyethylene (Low Density) (P.E.L)	0.15×10^3	0.5
Polyethylene (High Density) (P.E.H)	0.8×10^3	0.5
Galvanised Steel (G.S)	210×10^3	0.3
Asbestos Cement (A.C)	25×10^3	0.2
Cast Iron (C.I)	100×10^3	0.3
Ductile Iron (D.I)	170×10^3	0.3
Water	207×10^3	-

Table 7-5 Factor $ for uPVC Pipes

<table>
<tr><th>Pressure Class</th><th>Nominal outside diameter (mm)</th><th>uPVC Pipes Celerity, C, (m/s)</th><th colspan="2">Factor $</th></tr>
<tr><td rowspan="2">A
(P=0.6 MPa)</td><td>≤ 160</td><td>295</td><td colspan="2">30</td></tr>
<tr><td>>160</td><td>273</td><td colspan="2">28</td></tr>
<tr><td rowspan="2">B
(P=0.9 MPa)</td><td>≤ 160</td><td>355</td><td colspan="2">36</td></tr>
<tr><td>>160</td><td>331</td><td colspan="2">34</td></tr>
<tr><td rowspan="2">C
(P=1.2 MPa)</td><td>≤ 160</td><td>399</td><td colspan="2">41</td></tr>
<tr><td>>160</td><td>378</td><td colspan="2">39</td></tr>
<tr><td rowspan="2">D
(P=1.5 MPa)</td><td>≤ 160</td><td>444</td><td colspan="2">45</td></tr>
<tr><td>>160</td><td>419</td><td colspan="2">43</td></tr>
<tr><td rowspan="2">Steel Pipes Nominal diameter (mm)</td><td colspan="2">Heavy Series</td><td colspan="2">Light Series</td></tr>
<tr><td>Celerity, C (m/s)</td><td>Factor, $</td><td>Celerity, C, (m/s)</td><td>Factor, $</td></tr>
<tr><td>50</td><td>1,345</td><td>137</td><td>1,303</td><td>133</td></tr>
<tr><td>65</td><td>1,324</td><td>135</td><td>1,287</td><td>131</td></tr>
<tr><td>80</td><td>1,320</td><td>134</td><td>1,267</td><td>129</td></tr>
<tr><td>100</td><td>1,301</td><td>133</td><td>1,248</td><td>127</td></tr>
<tr><td>125</td><td>1,276</td><td>130</td><td>-</td><td>-</td></tr>
<tr><td>150</td><td>1,253</td><td>128</td><td>-</td><td>-</td></tr>
</table>

7.4.9 Practical Solutions to Water Hammer

7.4.9.1 Introduction

The surge pressure must be incorporated with the operating pressure in the design of the pipe. The following are some tools to reduce the effects of water hammer (Lahlou, 2003).

7.4.9.2 Valves

Water hammer often damages centrifugal pumps when electrical power fails. In this situation, the best form of prevention is to have automatically-controlled valves, which close slowly. (These valves do the job without electricity or batteries. The direction of the flow controls them.) Closing the valve slowly can moderate the rise in the pressure when the down-surge wave—resulting from the valve closing—returns from the reservoir.

Entrained air or temperature changes of the water can be controlled by pressure relief valves, which are set to open with excess pressure in the line and then closed when pressure drops. Relief valves are commonly used in pump stations to control pressure surges and to protect the pump station. These valves can be an effective method of controlling transients. However, they must be properly sized and selected to perform the task for which they are intended without producing side effects.

If pressure may drop at high points, an air and vacuum relief valve should be used. All downhill runs where pressure may fall very low should be protected with vacuum relief valves. Vacuum breaker-air release valves, if properly sized and selected, can be the least expensive means of protecting a piping system.

A vacuum breaker valve should be large enough to admit sufficient quantities of air during a down-surge so that the pressure in the pipeline does not drop too low. However, it should not be so large that it contains an unnecessarily large volume of air, because this air will have to be vented slowly, increasing the downtime of the system. The sizing of air release valves is, as mentioned, critical.

7.4.9.3 Pumps

Pump start-up problems can usually be avoided by increasing the flow slowly to collapse or flush out the voids gently. Also, a simple means of reducing hydraulic surge pressure is to keep pipeline velocities low. This not only results in lower surge pressures, but results in lower drive horsepower and, thus, maximum operating economy.

7.4.9.4 Surge Tanks

In long pipelines, surge can be relieved with a tank of water directly connected to the pipeline called a “surge tank.” When surge is encountered, the tank will act to relieve the pressure, and can store excess liquid, giving the flow alternative storage. Surge tanks can serve for both positive and negative pressure fluctuations. These surge tanks can also be designed to supply water to the system during a down-surge, thereby preventing or minimizing vapour column separation. However, surge tanks may be an expensive surge control device.

7.4.9.5 Water Hammer Estimation

To avoid complicated hydraulic surges, apexes on a rising main should be avoided or minimized. The best profile for a rising main is a concave profile along the longitudinal section of the rising main. The flow velocity in the rising main should be kept as low as possible since the surge pressure is a function of the flow velocity. Other important factors besides pipeline profile is pipeline length. The maximum surge for a single circular pipe without diameter change along its lengthcan be calculated using the eguation 7-6.

$$h = \frac{vc_p}{g} \qquad \textbf{Equation 7-6}$$

Where: v = average flow velocity, before rapid stoppage or closure in m/s,

c_p =surge wave velocity; which normally ranges between 600 - 1200 m/s depending on physical characteristic of the main

$$c_p = \frac{2c_w}{1 + \left(\frac{E_w}{E_p} x \frac{d}{t}\right)}$$

Where

c_w = celerity (speed) of the pressure wave in a column of water – 1425 m/s;
E_w = the bulk modulus of water =2230 N/mm^2 at 30^{o}C;
E_p = modulus of elasticity of pipe wall material in N/mm^2 (Table 7-4);
g = acceleration due to gravity (9.81m/s^2);
d = mean diameter of pipe (m); and
t = pipe wall thickness (m).

Abrupt pump stoppage can be avoided by fitting a pump with a heavy flywheel designed such that the pump still has some rotation at the end of the reflection time. A flywheel that is oil filled and can be adjusted to rotate the pump at a speed equal to or slower than that of the motor. This can be useful during the early years of a scheme when demand is les than maximum as it avoids the need for expensive variable speed motors. It has the added advantage that it can considerably reduce the magnitude of the surge wave.

Where ram pumps are installed, a pressure relief valve shall be installed on or after immediately the pump and before the valves mentioned above unless arrangements have been made by the manufacturer.

The surge tank design and location depends on the results of the hydraulic transient analysis as each system will have different characteristics. Detailed surge analysis using hydraulic modeling can be used to confirm if additional surge protection is required to prevent water hammer and column separation. After hydraulic modeling, surge protection and control features are identified basing on the system characteristics. These include pump control valves, check valves, surge relief valves, air valves and surge tanks. The two types of surge control tanks used in design are compressed- air- over water surge control system and bladder surge control system. Both tanks eliminate transient pressure in the system and can be used for protecting the transmission mains.

7.4.9.6 Air Chambers

Air chambers are installed in areas where water hammer is encountered frequently, and are typically seen behind sink and tub fixtures. Shaped like thin, upside-down bottles with a small orifice connection to the pipe, they are air-filled. The air compresses to absorb the shock, protecting the fixture and piping.

7.5. Air Valves

7.5.1 General

The peaks or crests in a pipe network and hence the number of air valves on a pipeline should be kept to the minimum. Also the quantities of air in a pipeline should be limited by preventing air entry into the pipeline.

The development of air pockets in a pipeline can cause major blockages to the flow of water in pipelines. Air pockets in a pipeline are caused mainly by:

i) Low pressure head in the pipeline; and
ii) High points in the pipeline, where the pressure head decreases relative to the hydraulic gradient as shown in Figure 7-4.

It is important to note that the high points are determined in relation to the hydraulic gradient for the pipeline, rather than in relation to the ground elevations.

Air valves serve three main purposes as follows:

i. To release air from a pipeline during the filling process, use large orifice valves;
ii. To release air from a pipeline during normal operation, use small orifice valves; and
iii. To allow air to enter into a pipeline in order to prevent the occurrence of a vacuum, use large orifice valves.

Air valves should be equipped with isolating (gate) valves, to facilitate the removal and repair of the valves.

7.5.2 Small Orifice Air Valves

On pipelines of diameters equal to or larger than 100 mm, small-orifice air valves should be placed at all high points relative to the horizontal. On smaller pipelines, small-orifice air valves should be placed only at “pronounced” high points and if at such points the air cannot alternatively be released through consumer service connections. In this context, a high point is considered to be “pronounced” if it stands more than 10 m higher than the low points immediately upstream or downstream of that point. A small-orifice air valve may also be required at the point where a steeply rising pipeline abruptly changes to follow a far milder slope. If a long stretch of pipeline has no pronounced high points a small-orifice air valve should also be installed at least after every kilometre. This is especially important when the pressure along the pipeline is decreasing, allowing dissolved air to form air bubbles. A valve orifice size of 2 mm is normally adequate for pipeline diameters of up to 300 mm.

7.5.3 Large Orifice Air Valves

Large- orifice air valves should be installed at pronounced high points on pipelines equal to or larger than 100 mm and at intervals of not more than 1 km. A valve inlet pipe diameter of 50 mm is normally adequate for pipeline diameters of up to 400 mm. At a point where a small-orifice air valve and a large-orifice air valve would coincide, a double-orifice air valve combining both a small-orifice air valve and a large orifice valve should be installed instead.

7.5.4 Single and Double Orifice Air Valves

Double orifice act as air release valves with sizes from 50 mm to 200 mm and allow release of air when water is flowing. The air release valve are fitted with small and large orifice. The air release valve is fitted with a cast-iron or stainless steel body, stainless steel or fibreglass balls, integral shut-off valve and flanged ends. The valve is equipped with an anti-shock facility. The valve is suitable for maximum pressure of 1600 kPa.

Single orifice air release valves for main water lines with sizes from 25 mm to 50 mm and allow air accumulated at specific points in the pipeline to leave when water starts flowing. The air release valve is fitted with a small orifice, cast-iron or stainless steel body, fibre glass or stainless steel ball float with threaded inlet. When the valve is installed, a shut-off valve is installed on the inlet side. The valve is equipped with an anti-shock facility. The valve is suitable for maximum pressure of 1600 kPa.

Single orifice double purpose air release valves for domestic water lines up to 15 mm. The air release valves are fitted with a stainless steel float, brass or cast steel body with an integral shut-off valve fitted. The valve is capable of withstanding a working pressure of 1000 kPa at 110 °C.

7.5.5 Alternatives to Air Valves

In cases where it is appropriate, air valves can be replaced with connections to private consumers or public water points or with rising branch pipelines. During the filling of a pipeline, washouts can also serve as air release points.

7.5.6 Side Vents

Side vents can be used where there is a possibility of allowing water from the pressure side to the suction side during pump shut off process. These vents have an electrical control mechanism to allow compressed air to influence slow opening and closing of the by-pass. This would enable correct control of the opening and closing time and the small dynamic pressure increases would ensure that the check valve closes gently. However, the reliability of the operation entirely depends on the control of the opening and closing time. This control process therefore requires an operator and there is energy loss through the reverse flow of the discharged water quantity. The diameter DN of the vent can be sized from around 0.3 up to 0.5D.

7.5.7 Controlled End Valve Closing

This is similar to the side vent but an additional backflow isolator is provided to reverse the pump speed not to rise above permissible limits. The additional by-pass line allows the check valve to close gently. However, operational reliability depends on the controlling of the closing time of the isolating valve and with parallel operation of several pumps on a common pressure main, additional control is required so that on opening or closing of one pump, the check valve closes swiftly. There is also energy loss through the discharged water mass.

7.5.8 Burst Disks

Burst disks are devices used to minimize damage when all other preventative mechanisms are unserviceable. Burst Disks are used where predetermined surge danger would rarely occur or when there are low water levels in the pumping system with several pumps operating simultaneously. These disks if installed alone will not be useful if dynamic differential pressure causes the disk to rupture on every pump shut-down operation. They should be mounted on an isolatable short pipe support so that only little water quantity is contained and accordingly, the moment effect of inertia is small. They are normally supplied in packs of five and the danger is the failure to re-order a replacement pack in time.

Reliance on air-valves only to protect pumping mains is not recommended as the risk of vandalism or inadequate routine maintenance can be catastrophic. The use of air-valves and burst disc devices is strongly recommended, and indeed such burst-disc devices are also recommended in conjunction with any air vessel requiring the periodic topping up by using an air compressor.

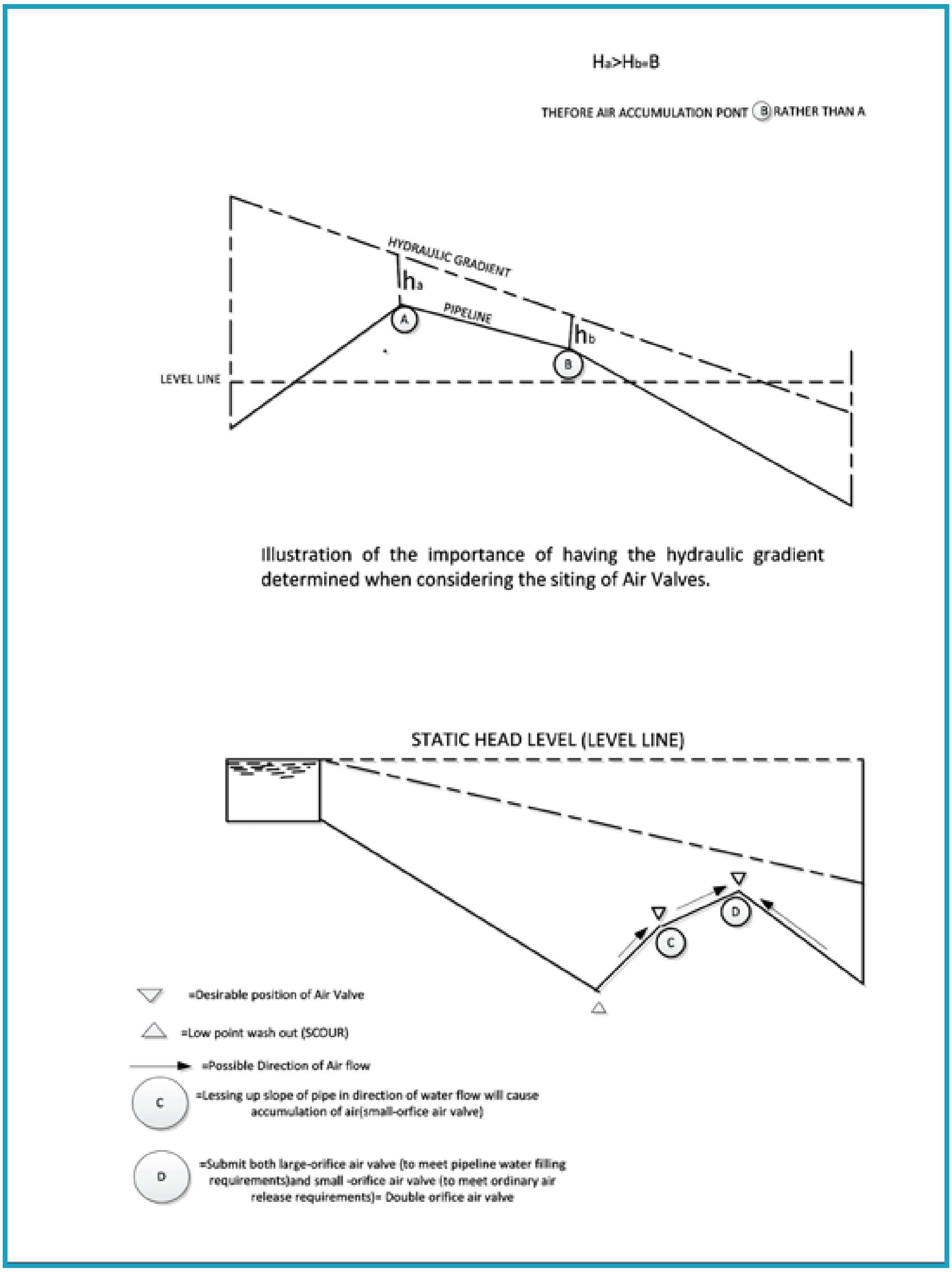


Figure 7-4: Siting of an Air Valve.

7.6 Washouts

7.6.1 General

Washouts are installed on pipelines to drain the pipe section especially during cleaning out of sediments in the pipe. They are provided at low points or valleys in the pipeline and should be kept to a minimum.

7.6.2 Location

Washouts should be placed only at “pronounced” low points or valleys on a pipeline. In this context, a low point is considered to be “pronounced” if the high points immediately upstream or downstream of that point stand more than 10 m higher than that point.

7.6.3 Washout Sizes

In a major pipeline, primary washouts may be installed to drain the majority of the length between section valves; secondary washouts of smaller diameter can then be used to empty un-drained low points. Sizes, particularly of primary washouts should be calculated according to the required drain down time, which should typically not be longer than one working shift. Factors for consideration are the number of washouts, head available and limits on discharge, access and resources. The drain down time is dominated by the low head available during the later stages. The washout diameters given below should allow the last 200 m length of a pipeline to be emptied in about one hour in typical situations.

Table 7-6 Washout Diameters

Size of the main	Size of the Wash out diameter
Up to 300 mm	80 mm
400 to 600 mm	100 mm
700 to 1000 mm	150 mm
1100 to 1400 mm	200 mm
1500 to 1800 mm	250 mm

If the normal shear stress of $10 N/m^2$ on the walls of a main pipeline, and the available pressure of 0.1 to 0.2 MPa are assumed, then the required sizes of washouts should be determined as follows.

$d = 0.6\ D$ (If the upstream and downstream sides of the pipeline are washed simultaneously.)

$d = 0.4\ D$ (If only one side of the pipeline is washed at a time)

Where:

d = diameter of the washout pipe in mm
D = diameter of the main pipeline in mm.

7.6.4 Washout Valves and Drains

Washout valves should be installed only on washout outlet pipes and not on the main pipelines. Open drains conveying water from the washouts away to suitable outfalls should be provided.

7.7 Firefighting

7.7.1 General

In fighting fires the flow rates from fire hydrants depend not on the water system design constraints, but on the type of equipment and the number of people involved in fighting the fire (DOH, 2009). As water system pressure decreases, the pump in the fire truck eventually begins to cavitate and is unable to deliver any substantial flow rate. At that point, the pump turns off. Design engineers should evaluate the potential that firefighting equipment may cause very low water system pressure. These low pressures may present a public health concern due to an increased risk for contamination from cross-connections and pathogen intrusion. Guidelines concerning provisions for fire-fighting have been presented in Chapter 2 – “Water Demand”.

7.7.2 Pipes

Fire hydrants do not require very high pipeline pressures, the more important requirement being the availability of adequate quantities of water in the pipelines. This requirement can be met by providing distribution pipelines of diameters of not less than 160 mm in high value commercial areas.

7.7.3 Fire Hydrants

In high value areas with high fire risks such as town centers and industrial areas, the distances between fire hydrants should be in the range of 65-100 m. An isolating section valve should be provided downstream of every fire hydrant.

7.8 Section Valves

7.8.1 Location

Section valves should be located in such a way that whenever required, selected sections of the distribution system can be closed off as appropriate. For small size and low pressure distribution pipelines, section valves are usually sluice (gate) valves. For high pressure and large diameter distribution pipelines exceeding 300 mm, butterfly valves should be used instead. On main distribution pipelines, section valves should be installed at distances of about 3 km in rural areas and about 1 km in urban areas.

All branch pipelines should have section valves at their points of connection to the main pipelines. Pumping mains should not have section valves outside pump houses. Wherever the siting of a section valve brings it adjacent to an air valve or a washout, the section valve should be installed upstream of and in the same chamber with the air valve or the washout. An isolating valve should be provided downstream of every fire hydrant.

7.8.2 Valve Chambers

A valve chamber should have size of at least 1,000 mm by 1,000 mm internally. Flexible pipes such as those made of uPVC should not be used inside the chamber. The chamber should have a lockable cover and adequate drainage facilities.

7.9 Break Pressure Devices

7.9.1 General

Break-pressure tanks and pressure-relief valves are used to keep pipeline pressures within the limits therefore making it possible to use lower pipe pressure classes in order to minimize pipeline costs. Break-pressure tanks and pressure-relief valves are also used to divide distribution systems into appropriate pressure zones. Wherever feasible, water storage reservoirs should be used also as break-pressure tanks. In general it is not recommended to use pressure-relief valves except in exceptional cases and where skilled operation personnel are available.

7.9.2 Break Pressure Tank Capacities

The capacity of a break-pressure tank should be large enough to give a retention period of at least 2 minutes.

7.9.3 Design Features of Break-Pressure Tanks

Break pressure tanks should have the following design features:

i) Be covered and have lockable access manhole covers;
ii) Have inlet pipes which end close to the tank floors, to prevent air entrainment by falling jets;
iii) Have overflow pipes, which are placed at least 50 mm above the normal top water level, and in

such a way that it is possible to see water overflowing; and

iv) Have ball float valves which are easily accessible from the access manholes, but which do not block the manhole.

7.10 Flow Direction Regulating Devices

7.10.1 Non-Return (Check) Valves

There are circumstances where it is necessary to ensure that water flows only in one direction. This applies mostly to consumer service connection pipes and pumping mains. The required "one-direction-flow" can be achieved by the use of non-return (check) valves. In a pumping main without a non-return (check) valve, water will flow backwards as soon as pumping stops. If a centrifugal pump is being used, it will not be able to start pumping again until it is filled with water, because the sump cannot remove air from its inside. In other words, the pump is not "self-priming". A non-return (check) valve on the suction pipe will prevent water from flowing backwards into the suction sump, and thus keep the pump always full of water, that is "primed".

In a long pumping main, with a high velocity of water flow and a high total static head, high water hammer (surge) pressures can develop when the pump stops and the non-return (check) valve closes. To reduce such water hammer (surge) pressures, the use of pipes of a higher pressure class than would normally be required or the installation of a surge tank or accumulator tank should be considered at the design stage. In most cases, it will be more economical to select a higher pressure class for part or the whole of the pipeline.

7.10.2 Stop Cocks

A stop cock is a section valve and a non-return valve combined in one unit. On every consumer service connection pipe, a stop cock should be provided to reduce the risk of drawing contaminated water from the consumer premises, backwards into the distribution system through the service connection pipe.

7.10.3 Pipe-Break Valves

To reduce the risk of flooding and water wastage when a pipeline breaks in large distribution system, self-closing security valves commonly called "pipe-break valves" can be installed close to the storage reservoirs. Pipe-break valves, which are usually of the butterfly type close when the velocity of water flow exceeds pre-determined values. Pipe-break valve should not be considered for use except in very large water supply schemes.

7.11 Marker Posts

Marker posts should be installed for the purpose of giving information on the pipeline at different locations. Marker posts should be provided along pipelines at a spacing of about 300 m. Marker posts should also be placed at all bends, Tee-junctions, section valves, air-valves, washouts, river and road crossings.

7.12 Anchor and Thrust Blocks

7.12 .1 General

Anchor and thrust blocks should be provided at horizontal and vertical bends, capped ends, changes of pipe sizes, tees and for pipelines laid in steep slopes exceeding 1:6. The sizes and types of anchor and thrust blocks required, will depend on the following factors:

i) Pressures in the pipelines;
ii) Sizes of the pipelines;

iii) Types of fittings used; and
iv) Type of soils traversed by the pipelines.

To determine required bearing area of anchor and thrust blocks, refer to Table 7-7 for safe soil bearing loads.

Table 7-7 Safe Soil Bearing Loads

Soil Type	Safe Soil bearing Load (kN/m^2)
Confined sand and gravel	200-500
Loam	150-300
Silt	130-160
Clay	90 -200
Silty/clayey soil	120-180

7.12 .2 Thrust Forces

Table 7-8 is a guide for designing thrust blocks for pipelines, and the data given is for thrust forces in pipelines with an internal pressure of 1.0 MPa. Anchor and thrust blocks should be designed for the highest pressure expected to occur in a pipeline. The highest pressure usually occurs during the pressure testing of the pipeline when, for example, uPVC pipelines are tested at 1.5 times their nominal working pressure. Generally, anchor and thrust blocks are not required where the thrust force is less than 0.05 x d kN, where d is the outside diameter of the pipeline in mm. Figure 7-5 shows the common ways of erecting anchor and thrust blocks.

Table 7-8: Resultant Forces on Bends

Outside Nominal Diameter (mm)	F= Axial Force (KN)	F= Resultant Force On Bends Of Angles (KN)					
		11.25°	22.5°	30°	45°	60°	90°
20	0.31	0.06	0.12	0.16	0.24	0.31	0.44
32	0.80	0.16	0.31	0.41	0.61	0.80	1. 13
50	1.96	0.38	0.76	1 01	1.50	1.96	2. 77
63	3.12	0.61	1.22	1.62	2.39	3.12	4.41
90	6.36	1.25	2.48	3.29	4.87	6.36	8.99
110	9.50	1.86	3.71	4.92	7.27	9.50	13.44
160	20.11	3.94	7.85	10.41	15.39	20.11	28.44
225	39.76	7.79	15.51	20.58	30.43	39.76	56.23
280	61.58	12.07	24.03	31.88	47.13	61.58	87.09
315	77.93	15.28	30.41	40.34	59.65	77.93	110.21

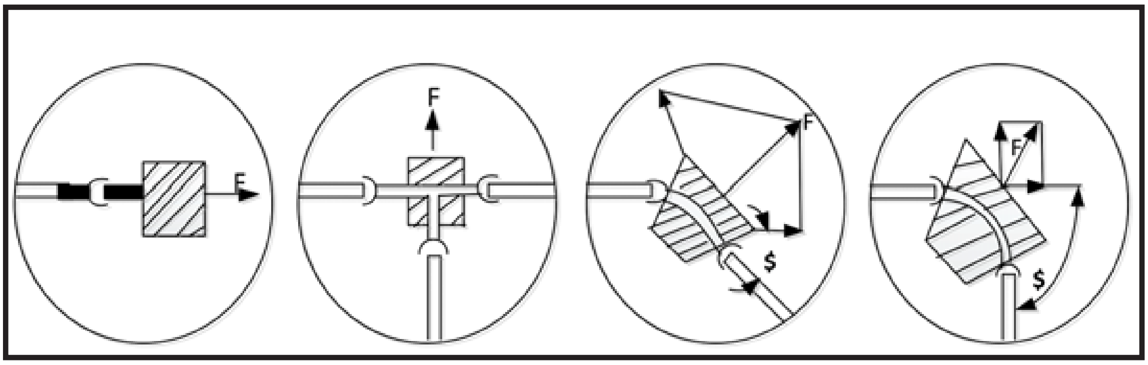


Figure 7 5: Anchor and Thrust Blocks.

7.14 Leak Detection

To facilitate leak detection in large distribution systems, the design should include small diameter by-passes downstream of the storage reservoirs, as illustrated in Figure 7-6. The by-passes should be fitted with water meters for the accurate measurement of small water flows at night (called the "night line"), the largest proportion of which represents leakage.

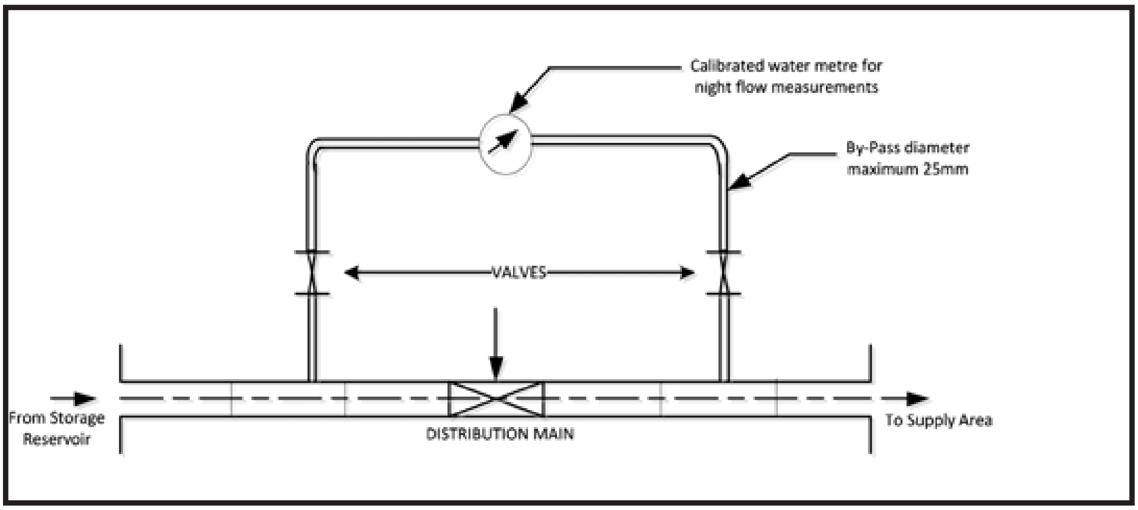


Figure 7-6: By-pass for Leak Detection.

7.14 Consumer Service Connections

7.14.1 General

The number and types of water consumer service connections have considerable influence on the design of water distribution systems. A distinction can be made between the following main types consumer service connections:

i) Private Connections (House Connections and Yard Connections); and
ii) Public water points (Stand posts).

7.14.2 Private Connections

Private connections include house connections and yard connections to domestic, institutional, commercial and industrial consumer premises. A house connection is a consumer service pipe connected into a building which has internal plumbing and drainage systems. For a yard connection, the consumer service pipe is connected to a water tap which is placed in the yard outside the building, because it does not have internal plumbing and drainage systems. A distribution system should be designed to cater for a definite number of private connections.

The sizes of consumer service connection pipes for private connections should be calculated in the same way as those for public water points. Normally, the pipes should not be less than 20 mm in diameter and not longer than 100 m in length. The sizes of the stop cocks, water meters and connection pipes should be determined individually for each case, but they should not be less than 20 mm in diameter. Normally, a water discharge rate of about 15 l/min is required in consumer service connection pipes. All private connections should be metered.

7.14.3 Public Water Points

In rural areas, public water points (stand posts) should be sited in such a way that the maximum walking distance for 90% of the users does not exceed 1.5 km. Also, the number of users per water point (stand post) should normally be restricted to 200 persons. In urban centers, the maximum walking distance to public water points (stand posts) should not exceed 250 m and the number of users per water point should be restricted to 150 persons. However, the local water collection habits, the numbers of private connections in the area and other relevant factors should be considered before siting public water points (stand posts). The final siting of public water points (stand posts) should be done in collaboration with local consumer communities and authorities.

Public water points (stand posts) should be sited on high ground to facilitate the drainage of spilt water, and also to make the points serve as air-outlets at peaks in distribution pipelines. The points should be fenced off to restrict entrance. Public water points (stand posts) should be designed and constructed according to the standard drawings approved by the Directorate of Water Development. Every water point should be provided with a stopcock and a meter. Proper drainage facilities should be provided for water points.

7.14.4 Water Meters

A water meter is a device used to measure the volume of water usage. In Uganda, water meters are used to measure the volume of water used by residential and commercial buildings that are supplied with water by a public water supply system. Water meters can also be used at the water source, and throughout a water system to determine flow through that portion of the system. Water meters measure flow in Cubic metres (m^3) on a mechanical or electronic register. Some electronic meter registers can display rate-of-flow in addition to total usage.

There are several types of water meters in common use. Selection is based on different flow measurement methods, the type of end user, the required flow rates and accuracy requirements. There are two major methods of flow measurement in use including displacement and velocity with sub-technologies within each of them. Common displacement designs include oscillating piston and rotating disk meters. Velocity-based designs include single- and multi-jet meters and turbine meters. There are also non-mechanical designs, for example electromagnetic and ultrasonic meters.

In addition to the more common types of meters, there are meters designed for special uses. Most meters in a typical water distribution system in Uganda are designed for cold potable water only. There are special water meters manufactured for specific other uses including hot water meters designed with special materials that can withstand higher temperatures.

Water meters are generally owned, read, and maintained by a public water provider such as National Water and Sewerage Corporation or private water operators managed by the Ministry of Water and Environment. In some cases, an owner of an apartment complex or commercial building may want to share the cost of the bill among the tenants. In this case, private meters may be purchased to separately track usage of each unit in what is called sub metering.

Prepaid meters are highly recommended for public water points that operate as kiosks. Public water points can have one or more taps. To determine the delivery rates of taps at different supply pressures, the following formula can be used:

$$q_{act} = q_{nom}\sqrt{10H_{tap}}$$

Equation 7-7

q_{act}	=	the actual delivery rate of a tap, l/min
q_{nom}	=	the nominal delivery rate of a tap, l/min at a pressure of 0.1 MPa (10 m of water head) before the tap
H_{tap}	=	the available pressure just before the tap, in Mpa

Typically, the q_{nom} of a 12 mm tap and a 20 mm tap are 13 l/min and 25 l/min respectively. Assuming an efficiency of 80%, it follows from the foregoing exposition that a supply pressure of 5 m head of water will result in a delivery rate of 8 l/min for a 12 mm tap, and 14 l/min for a 20 mm tap. Assuming the same efficiency, a supply pressure of 60 m head of water (the maximum recommended supply pressure) will result in a delivery rate of about 75 l/min for a 12 mm tap, and 140 l/min for a 20 mm tap.

The use of taps at high delivery capacities can result in large quantities of water being wasted even when the taps are left open for only short periods of time. Therefore, taps should be used at delivery capacities restricted to a maximum of 30 l/min. From the foregoing considerations, together with the required delivery rates and supply pressures a consumer service connection pipe should have a diameter of not less than 40 mm, and a length of not more than 100 m.

$$h_{wm} = 0.1\times\left(\frac{q_{act}}{q_{nom}}\right)^2 \quad \text{MPa}$$

Equation 7-8

h_{wm}	=	head loss through the meter in MPa
q_{act}	=	the actual flow through the meter in m^3/h
q_{nom}	=	the nominal rating of the water meter in m^3/h at a pressure of 0.1 MPa

The calculated delivery capacity of a public water point should preferably lie in the range of 50-80% of the nominal capacity of its water meter.

7.15 Bulk Water Transfer

7.15.1 General

A system of canals known as aqueducts can be used for bulk water transfer to high water demand areas from areas where there is adequate water. In case of aqueducts, they should be routed in such a way as to allow delivery of water by gravity whenever possible. Apart from topography, other factors such as distribution methods, storage facilities and water demands must be considered. Pipelines can also be used for bulk water transportation although the cost of construction, operation and maintenance may be high.

The objective of bulk water transfer is to make available adequate quantities of water all year round for multiple uses such as irrigation, livestock, aquaculture, industrial use, commercial activities and drinking. Implementing bulk water transfers is seen as a contribution to achieving the MDGs. Four priority areas with acute water shortage have so far been identified in Uganda for consideration of bulk water transfer. These include Bukanga and Nyabushozi in Western Uganda; Kakuuto, Kooki and Kabula in Southern Uganda; Nakasongola in Central Uganda and Nakapiripirit in North Eastern Uganda.

The large scale transfer of water can have far reaching implications in both water supplying and receiving catchments. In addition issues of inequity between the receiving and supplying communities need careful consideration in planning bulk water transfers (CAPRi, 2008). Environmental and social considerations must be taken into account before large scale water transfers are implemented so that conflicts within and outside the beneficiary communities over sharing water are foreseen and mitigated. The ESIA should be carried out as specified in Chapter 8 – "Environmental and Social Impact Assessment" and the relevant regulations of the National Environment Act.

Bulk water transfers can be considered from Mt. Elgon, Mt. Ruwenzori, Lake Kyoga and other major lakes and rivers. Bulk water transfers are possible for rivers of a transboundary nature and lakes such as Lake Victoria and River Nile; however, in transboundary water resources, the legislation governing the use and management of these waters must be adhered to. Further information on bulk water transfers can be obtained from the Water for Production Subsector Strategy of the MWE.

7.15.2 Bulk Water Transfer from Mountains

All the major rivers of the world have their headwaters in highlands and more than half of humanity relies on the freshwater that accumulates in mountain areas (Liniger and Weingartner, 1998). This punctuates the importance of water from mountains and its recognition as a key resource that should be considered for supply. Water from mountains is fresh, free of contamination and can always be supplied by gravity.

There are many reasons why water from the mountains should be given particular focus:

i) **High Precipitation Levels.** Mountains form a barrier to incoming air masses. They force air masses to rise, cool thereby triggering precipitation. In semi-arid and arid regions, only highlands have sufficient precipitation to generate runoff and groundwater recharge. Mt. Elgon and Mt. Ruwenzori have the highest rainfall occurrence of over 2,000 mm annually in Uganda.

ii) **Storage and Distribution of Water to the Lowlands.** Water captured at high altitudes flows under gravity via stream networks or groundwater aquifers to the lowlands, where the demand from population centers, agriculture and industry is high. In humid areas the proportion of water generated in the mountains can comprise as much as 60% of the total freshwater available in the watershed, while in semi-arid and arid areas this proportion is generally over 90% (Liniger and Weingartner, 1998). The areas of Kapchorwa, Bukwo, Sironko, Manafwa, Mbale and Tororo rely on water supplies from Mt. Elgon which rises up to 4,321 m above sea level. The areas of Kasese, Kabarole and Bundibugyo rely on water supplies from the snow-capped Mt.Ruwenzori which rises up to 5,109 m above sea level.

iii) **Source of Hydropower.** While mountains provide water supplies they can concurrently be a source of hydropower tapped from water falling over large heights. This can be seen as an additional economic value of water from mountains.

Mt. Elgon and Mt. Ruwenzori have catchments that fall under the category of water from mountains. The average annual rainfall in these mountains is above 2,000 mm representing the highest rainfall in Uganda. These waters can be exploited to serve the areas in the plains below them and also for generation of hydropower. Water from Mt. Elgon can be considered for supply to the water scarce areas in the east and north eastern Uganda such as Teso and Karamoja. Mt. Ruwenzori may be considered for the areas in Western Uganda.

7.15.3 Bulk Water Transfer from Lakes and Rivers

When considering bulk water transfers from lakes and rivers the impact of abstracted water to the receiving and supplying catchments must be carefully studied. In addition, the prevailing legislation on water resources has to be complied with. The National Water Policy 1999, the Water Act, the Water Resources Regulations and National Environment act must be consulted before any project is implemented. All major lakes and rivers in Uganda can be considered for bulk water transfer. For the identified water scarce areas of East, North Eastern and Central Uganda, Lake Kyoga is seen as the main source in this category.

Bulk water transfers from lakes and rivers are a useful source especially for irrigation, aquaculture, livestock and industries where water does not have to be treated before supply. In this case the considerations to be made are volumes of water abstracted, transmission route, environmental and socio impacts and provision for storage facilities.

7.15.4 Bulk Water for Industry and for Production

Consideration may be made for supply of water that is not potable for large scale industrial activity and agriculture. Water for cooling, aquaculture and irrigation may be supplied through piped networks that are parallel or adjacent to the treated water supplies targeting the large consumers. Billing of these supplies should be done at a lower tariff than for potable water.

7.15.5 Bulk Water Metering

Bulk flow meters are used at intakes, treatment works and in the distribution sytems at reservoirs and bulk supply. They are installed for purposes of monitoring large flows of water for water system management and commercial billing purposes. They are normally equipped with helical vanes with pulse outputs for operation with various auxiliary equipment. Different body lengths and material types are available to meet all requirements.

Combination meters are manufactured for installations where wide variation in flow can be expected, such as in multi story building, hospitals, schools, offices and other places where both low and high flows can occur due to several consumptions users. These wide flow ranges are measured by using a built-in changeover valve together with small residential meters and large bulk meter.

Large commercial single jet meters are also available which have a low flow capability,which makes them ideal for revenue collection.Electromagnetic water meters are also available which are designed for measuring bulk flows in a wide range of applications including irrigation management of agricultural land.All bulk meters should be tested to ensure that they meet approved standards.

WATER PUMPING

8.1 Pump Selection

8.1. General

Pumps are hydraulic machines which convert mechanical energy (imparted by rotation) into water energy used in lifting (pumping) water to higher elevations. The mechanical energy is provided by electrical power (motor) or diesel, gas or steam prime movers using either vertical or horizontal spindles. There are two main pump types used in Uganda as outlined in Table 8-1.

Table 8-1: Most Commonly Used Pump Types in Uganda.

Main Type	Sub-type	Specific types
Rotodynamic	Centrifugal	Single-stage
		Multi-stage shaft driven
		Multi-stage submersible
	Axial – and mixed	Axial flow
		Mixed flow
Positive displacement	Reciprocating	Suction (shallow well)
		Lift (deep well)
	Rotary	Helical Rotor

8.1.2 Rotodynamic Pumps

8.1.2.1 Introduction

In the rotodynamic-type pump, water while passing through the rotating element (impeller) gains energy which is converted into pressure energy by an appropriate impeller casing.

8.1.2.2 Centrifugal Pumps

In the form of tall, slender, deep-well submersibles, they pump clear water using impellers to displace water by momentum, rather than by positive mechanical travel. When rotated at sufficient speeds, impellers convert the velocity energy of the water leaving the impeller periphery into pressure energy. The capacity of the centrifugal pump is greatly influenced by the pressure it works against, and also by the speed, form and diameter of its impeller. Low speed centrifugal pumps wear less and last longer than high speed pumps. Generally, speeds selected for raw water pumps should be limited to a maximum of 1500 rpm.

Pump efficiency should always be maximized by choosing a pump which will operate close to the peak of its efficiency curve. To determine the operating (duty) of a pump, system head losses should be calculated as accurately as possible. The steeper the pump characteristics curve is, the less will the actual capacity of the pump deviate from the wanted capacity, the actual pumping head differs from that assumed at design stage. For this reason, pumps with steep characteristic curves are preferred. Suction head must never exceed head losses, as the operating (duty) point of the pump will be very difficult to determine correctly.

The pump manufacturers should always be consulted before the final choice of a pump is made and the size of the engine or motor is selected.

The characteristics and applicability for the different types of centrifugal pumps include:

i) Single-stage: the usual depth range is 20 – 35 m. it requires skilled maintenance; not suitable for hand operation, powered by engine or electric motor;
ii) Multi-stage shaft-driven: the depth range is 25 – 50 m. it requires skilled maintenance; the motor is accessible, above ground; alignment and lubrication of shaft critical; it has a capacity range of 25 – 10,000 l/min; and
iii) Multi-stage submersible: the depth range is 30 – 120 m. its operation is smoother but maintenance is difficult; repair to motor or pump requires pulling unit from well; it has wide range of capacities and heads; subject to rapid wear when sandy water is pumped.

8.1.2.3 Axial- and Mixed-Flow pumps

An Axial-flow propeller pump consists of propeller which thrusts rather than throws the liquid upward. Impeller vanes for mixed-flow centrifugal pumps are shaped to provide partial throw and partial push of the liquid outward and upward. Axial- and mixed-flow designs can handle large capacities but only with reduced discharge heads. They are constructed vertically. Axial flow pumps are used mostly for high-capacity and low-lifting pumping. They can pump water containing sand or salt. Axial flow pumps are the nominal choice for high-volume, low head raw water pumping. They are available in a wide range, capacities and sizes. They are usually installable to a depth range of 5 – 10 m.

Specific Speeds

Specific speed (N_s) is the parameter which characterizes the rotodynamic pumps more explicitly and is given by:

$$N_s = \frac{NQ^{0.5}}{H_m^{0.75}}$$

Equation 8-1

Where Q = the discharge (l/s)
H_m = the total (manometric) head (m)
N = the rotational speed (rev/min)

The specific speed for various Rotodynamic Pump types are shown in Table 8-2:

Table 8-2: Specific Speeds for Rotodynamic Pumps.

Type			Specific speed
Radial flow	i.	slow speed	300-900
	i.	medium speed	900-1,500
	i.	high speed	1,500-2,400
Mixed flow			2,400-5,000
Axial flow			5,000-15,000

8.1.3 Positive Displacement Pumps

8.1.3.1 Reciprocating Pumps

The reciprocating pump utilizes the energy transmitted by a moving element (piston) in a tightly fitting case (cylinder). Frequently in reciprocating pumps, a piston or plunger is used in a cylinder, which is driven forward and backward by a crankshaft connected to an outside drive. The reciprocating pumps can be divided into the two main categories; suction pumps and lift pumps.

8.1.3.2 Suction Pumps

Suction pumps are used in shallow wells. In a suction pump, the pump element (cylinder and plunger) is positioned above the water level, usually within the pump stand itself. A suction pump relies on atmospheric pressure for its operation. They lift water through a vacuum (sucking) action. All the moving parts are above the ground. Typically in the form of cast iron pumps, but also come in different forms such as the plastic Rower pump and diaphragm pumps. The suction pumps can be installed up to a depth of 7 m.

8.1.3.3 Lift Pumps

Lift pumps are used in shallow wells. In a lift pump, the pump element (cylinder and plunger) is located below the water level in the well. Lift pumps create lift of the water, most commonly using a piston with leather, rubber or plastic washers (cup seals) located in a pump cylinder below the water level. The piston travels in an up and down motion at the pump head (direct action), a lever type handle, or a circular motion handle. Other mechanisms include spiral or helical stainless steel rotors encased in a rubber stator in the cylinder, and rubber diaphragms actuated hydraulically.

The depth ranges of the lift pumps are as follows:

i) Low lift: up to 25 m
ii) Intermediate lift: 25 to 50 m
iii) Deep-set: 50 to 90 m

8.1.3.4 Hand Pumps

Hand pumps are the most common used reciprocating pumps and, in most cases, the only economically feasible water lifting device for community needs (UNICEF, 1999). Yield depends on the depth and design, normally in the range of 600 to 1,500 litres per hour during constant use. The most important design criterion for a hand pump is its maintainability. Some typical maintenance programmes for Hand pumps include: periodic lubrication of above-ground components, replacement of washers and seals, replacement of plastic bearings, occasional replacement of individual riser mains. The maximum pumps (lifts) for comfortable operation of hand pumps are shown in the table below.

Table 8-3: Maximum Heads (Lifts) for Hand pumps.

Cylinder Diameter (mm)	Head/lift (m)
50	Up to 25
65	Up to 20
75	Up to 15
100	Up to 10

The maximum discharge from a hand pump can be estimated using the following formula:

$$Q = \frac{460 \times e}{H} \qquad l/min$$

Equation 8-2

Where:

Q	=	maximum discharge from the hand pump (l/s)
H	=	pumping head (m)
e	=	pumping efficiency (a value between 0 and 1)

8.1.3.5 Rotary Pumps

Positive displacement rotary pumps are pumps work on the principle of rotation. The rotation of the pump creates vacuum which draws in the liquid. The need to bleed the air from the lines manually is eliminated in rotary pumps because the air from the lines is naturally removed. Positive displacement rotary pumps also have their weaknesses. Because of the nature of the pump, the clearance between the rotating pump and the outer edge must be very close, requiring that the pumps rotate at a slow, steady speed. If rotary pumps are operated at high speeds, the fluids will cause erosion, and thereby showing signs of enlarged clearances, which allow liquid to slip through and detract from the efficiency of the pump.

Helical rotor pumps are the most commonly used type of rotary pump. A helical rotor pump consists of a single thread helical rotor which rotates inside a double thread helical sleeve, the stator. It is the meshing helical surfaces which force water up to create a uniform flow. Water delivery by rotary pumps is continuous and therefore smoother. However, internal losses in rotary pumps are normally higher through slip (internal leak-back). Slip increases with increasing pressure, making rotary pumps unsuitable for use in high pressure systems.

8.1.4 Considerations for Pump Selection

The factors which are normally considered in the selection of a pump include:

i) Depth to the water level and its seasonal variations;
ii) Pressure ranges needed for adequate water supply;
iii) Heights through which water has to be lifted, both below and above the pump;
iv) Pump location;
v) Pump durability and efficiency; and
vi) Pump supplier after sale services.

The type of pump selected for a particular installation should be determined on the basis of the following fundamental considerations:

i) Yield of the well or water source;
ii) Daily needs and instantaneous demand of the users;
iii) The “usable water” in the pressure or storage tank;
iv) Size and alignment of the well casing;
v) Total operating head pressure of the pump at normal delivery rates, including lift and all friction losses;
vi) Difference in elevation between ground level and water level in the well during pumping;
vii) Availability of power;
viii) Ease of maintenance and availability of replacement parts;
ix) First cost and economy of operation;
x) Reliability of pumping equipment; and
xi) Pump start-up problem and time.

8.1.5 Well Casing and Screen

To keep loose sand and gravel from collapsing into the borehole, it is necessary to use well casing and screen. The screen supports the borehole walls while allowing water to enter the well.Unslotted casing is placed above the screen to keep the rest of the borehole open and serve as a housing for pumping equipment. Since the well screen is the most important single factor affecting the efficiency of a well, it is sometimes called the “heart of the well”.

8.1.5.1 Screen Design

Well screens should have as large a percentage of non-clogging slots as possible, be resistant to corrosion, have sufficient strength to resist collapse, be easily developed and prevent sand pumping (Driscoll, 1986). PVC pipe, which is available, relatively cheap, corrosion resistant, lightweight, easy to work with and chemically inert should be used.

Using a hack saw, cut slots in the plastic casing which are as long and close together as possible. Slots should be spaced as close together as possible vertically and should extend about a fifth the circumference of the pipe.There should be 3 even rows of slots extending up the pipe separated by 3 narrower rows of solid, uncut pipe (for strength).

75mm diameter casing and screen should be inserted into a 150mm borehole to allow creation of an effective 30mm thick filter pack (this is especially important where the aquifer is composed of very fine materials). However, in the case of unavailability of the 75mm diameter casing, carefully centered and filter packed 100mm screen can be used in the 150mm borehole allowing creation of an effective 25mm thick filter pack. Larger diameter screens make the filter pack ineffective and do NOT significantly increase well yield. For example, moving from a 100 to 150mm screen will increase yield by 3 percent or less.

Table 8-4: Information on Pump Selection

Pump Type	Practical suction lift, ft. (m)	Usual well-pumping depth, ft. (m)	Usual pressure heads, ft. (m)	Advantages	Disadvantages	Remarks
Positive displacement				1. Positive action 2. Discharge against variable heads 3. Pumps water containing sand and silt 4. Especially adapted to low capacity and high lifts	1. Pulsating discharge 2. Subject to vibration and noise 3. Maintenance cost may be high 4. may cause destructive pressure if operated against closed valve	1. best suited for capacities of 5-25 gpm against moderate to high heads 2. adaptable to hand operation 3. can be installed in very small diameter wells (2-in casing) 4. pump must be set directly over well (deep well only)
Shallow well						
(Reciprocating)						
Deep well	22-25	22-25	100 – 2200			
(Reciprocating)	(6.71 – 7.62)	(6.71 – 7.62)	(30.48 – 670.56)			
				1. smooth, even flow 2. pumps water containing sand and silt 3. pressure on system is even and free from shock 4. Low starting torque 5. usually reliable and good service life	1. Loses prime easily 2. Efficiency depends on operating under design heads and speed	
Centrifugal						
Shallow well						
Straight centrifugal (single stage)	20 ft. (6.10) max.	10 – 22 (3.05 – 6.10)	100 – 150 (30.48 – 45.72)			

Pump Type	Practical suction lift, ft. (m)	Usual well-pumping depth, ft. (m)	Usual pressure heads, ft. (m)	Advantages	Disadvantages	Remarks
Regenerative vane turbine type (single impeller)	28 (8.53) max.	28 (8.53)	100 – 200 (30.48 – 60.96)	1. same as straight centrifugal except not suitable for pumping water containing salt or silt 2. self-priming	1. same as straight centrifugal except maintains priming easily	
Deep well						
Vertical line shaft turbine (multi stage)	Impellers submerged	50 – 300 (15.24 – 91.44)	100 – 800 (30.48 – 243.84)	1. same as shallow well turbine 2. all electrical components are accessible, above ground	1. efficiency depends on operating under design head and speed 2. requires straight well large enough for turbine bowls and housing 3. lubrication and alignment of shaft critical 4. abrasion from sand	

Table 8-5: Information on Pump Selection (continued)

Pump Type	Practical suction lift, ft. (m)	Usual well-pumping depth, ft. (m)	Usual pressure heads, ft. (m)	Advantages	Disadvantages	Remarks
Submersible turbine (multistage)	Pump and motor submerged	50 – 400 (15.24 – 121.92)	50 – 400 (15.24 – 121.92)	1. same as shallow well turbine 2. easy to frost-proof installation 3. short pump shaft to motor 4. quiet operation 5. well straightness not critical	1. repair to motor or pump requires pulling from well 2. sealing of electrical equipment from water vapour critical 3. abrasion from sand	1. 3500 – rpm models, while popular because of smaller diameters or greater capacities, are more vulnerable to wear and failure from sand and other causes
Jet (ejector):						
Shallow well (4.57 – 6.10)	15 – 20 (4.57 – 6.10) Below ejector	80 – 150 (24.38 – 45.72) Below ejector	80 – 150 (24.38 – 45.72)			
Deep well	15 – 20 (4.57 – 6.10) Below ejector	25 – 120 (7.62 – 36.58), 200(60.26) Below ejector	80 – 150 (24.38 – 45.72)	1. same as shallow well jet 2. well straightness not critical	1. same as shallow well jet 2. lower efficiency, especially at greater lifts	1. the amount of water returned to ejector increases with increased lift – 50% of total water pumped at 50 ft. (12.54 m) lift and 75 % lift at 100 ft. (30.48 m) lift

8.2 Performance Parameters for Pumps

8.2.1 General

When specifying centrifugal pumps it is important to understand the various parameters that effect pump performance and their relationship with one another.

The parameters which affect pump performance include: speed, impeller diameter and number of Impellers

i) Speed: the effect of doubling impeller speed is that, increases power consumed and pressure by a factor of 23 = 8 and 22 respectively;
ii) Impeller diameter: Impeller diameter effects pump performance in a similar way to speed. A 10% increase of impeller diameter increases power consumed and pressure by 33% and 21 % respectively; and
iii) Number of Impellers: Impellers in series increases pressure, though has no effect on flow. This is the effect of a multistage pump. Adding impellers in parallel increases flow though has no effect on pressure. This is the effect of two pumps connected in parallel.

Typically a pump curve will provide the following information.

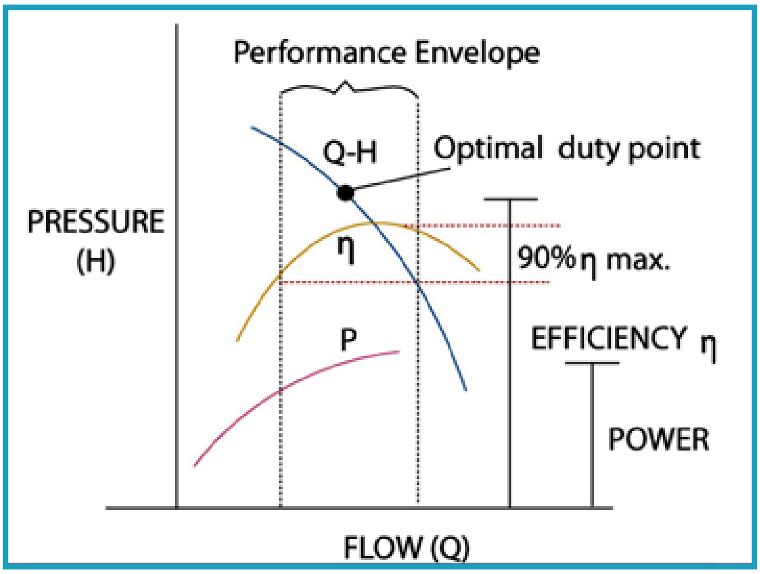


Figure 8-1 A Typical Centrifugal Pump Performance Curve

Three plots are given against flow – Pressure (or Q-H curve), Efficiency (ŋ) and Power absorbed.

i) Pressure: at zero flow the pump will provide its maximum pressure (closed head pressure). At zero head the pump will provide its maximum flow;
ii) Efficiency (ŋ): the efficiency curve is the plot of overall efficiency against flow. The pump's optimal duty point is that at which peak efficiency occurs and is usually around the midpoint of the curve. The optimal performance envelop is the flow range which is greater than 90% of the pump's maximum efficiency and applications should be within this envelope. Efficiency drops considerably at high pressures and high flows and specifying a pump to operate in these sections of a curve must be avoided; and

iii) Power: the power curve is a plot of power consumed against flow. It is important to note that, the maximum power consumption of a pump occurs at high flows/low pressures. Usually power consumed at high pressures is lower. When coupling motors to pumps it is important to ensure that the power consumed at open delivery is less than the motor size or else motor failure may occur.

There are three principal performance parameters relating to pump selection: flow (or capacity), total delivery head and suction lift.

8.2.2 Capacity

Required capacity, measured in flow/time is determined by one of two factors:

i) If there is storage capacity, it is related to total daily demand. Daily demand must first be estimated and then the hourly requirement calculated by dividing the daily demand by the number of hours the pump is required to work; and

ii) If there is direct supply pump capacity should be related to peak hourly demand. This would be appropriate in irrigation or pressure systems.

Capacity is measured in various units including liters per second, liters per minute and cubic meters per hour.

8.2.3 Total Head

8.2.3.1 Introduction

There are three principal components to total head of importance when specifying a pump: static head, dynamic head (friction loss) and pressure head.

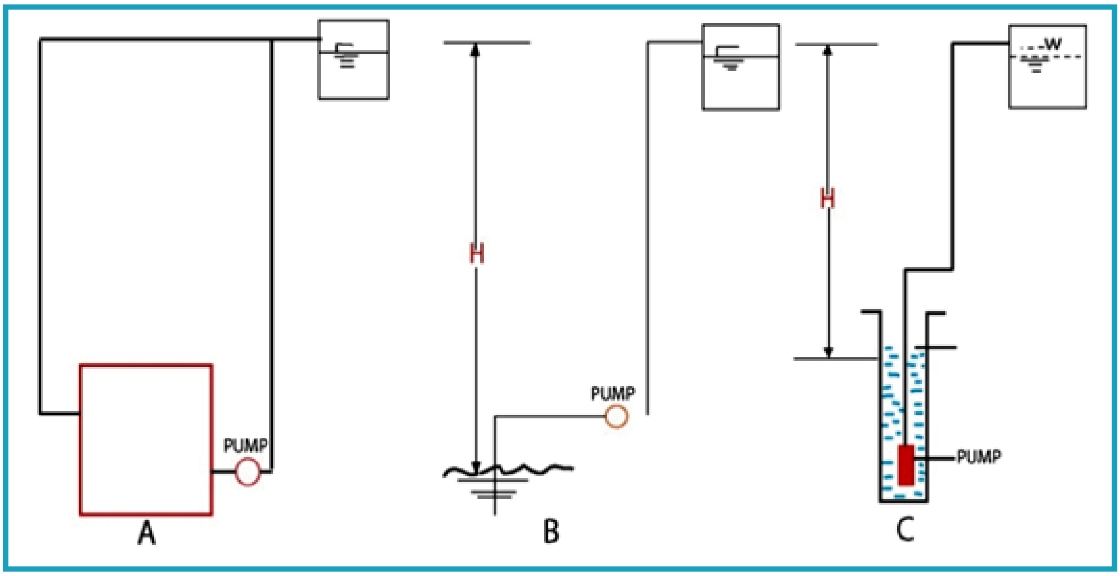


Figure 8-2: Static Head Conditions

8.2.3.2 Static Head (H)

Static head is the vertical linear distance between the level of the water being pumped and either the delivery outlet or the reservoir water level, whichever is higher (see A & B of Figure 8-2). Of great importance to note is that it is not necessarily the distance between the pump itself and the delivery point. This has particular reference to submersible pumps where the level the pump is set in the water does not determine static head. It is determined by the pumping water level (see C).

8.2.3.3 Dynamic Head

The only important component of dynamic head is pipe friction, this being determined by water velocity in the delivery pipe. The higher the velocity the higher the friction loss and it is important to match the pump to the pipeline. Friction loss values for GI and PVC pipes are given in Appendix.

Some important points to note when matching pumps and pipelines are:

i) Friction losses are considerably lower in PVC pipes than GI ones. For long pipelines the use of PVC will therefore reduce pump size and energy consumed;
ii) Piping can be considerably more expensive than the pumping installation and a pipe size smaller matched to a pump size larger can reduce the investment cost. Running costs will be higher though; and
iii) Total head reduces up the pipeline and lighter duty pipes can be used towards the system's delivery point.

Total friction loss for a pipeline (HF) = F x L/100

Where:

F = Friction loss given for a particular .flow in a specified pipe size (m per 100m pipe length).

L = Pipe length (m).

8.2.3.4 Pressure Head

When delivering to an open outlet pressure at the delivery point is zero and so in most water supply installations this is not a factor in total head calculations. However when pressure delivery is required, for instance, for fire installations or irrigation nozzles, the required pressure at the nozzle must be included when calculating total head.

In order to find the total head required on a pump, static head plus dynamic head plus friction head must be added. This can be done graphically as follows. From figure 8-3 below, pump 3 or pump 4 can be selected, depending upon required pump capacity.

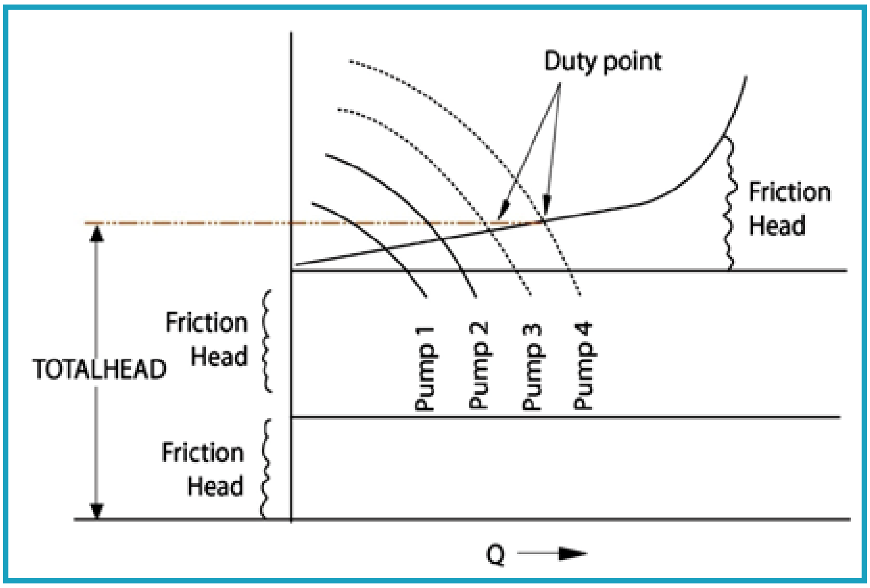


Figure 8-3: System Head Curve

8.2.4 Suction Lift

8.2.4.1 General

A suction lift is the vertical distance from the water level to the centre of the pump. The pressure of the atmosphere will support a column of water about 10.36 m high at sea level and it therefore follows that, maximum height to which water can be theoretically lifted by creating a perfect vacuum in the suction pipe is 10.36 m. Pumps lift water by generating a partial vacuum which permits the atmospheric pressure to force the water up the suction pipe. It is not however, practically possible for the pumps to raise water through that height as the pumps can never create a perfect vacuum, because of the frictional losses. Therefore, a pump is not used for a total suction height of more than 7.31 m and which too generally varies from 4.57 – 6 m according to the make of the pump. For design purposes it is usual to assume 4.88 m for centrifugal pumps and 6 m for reciprocating pumps.

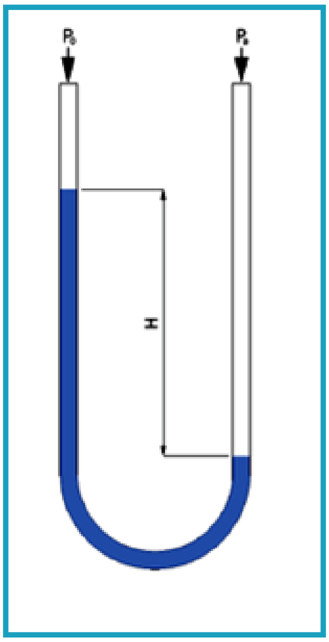


Figure 8 4 The Manometer

Referring to Figure 8 -4 , the result for the application of Bernoulli's equation between the sump water level and the pump (impeller) inlet indicates that the pressure at the pump inlet, p_s, is below the atmospheric pressure, p_a, and if this negative pressure exceeds the vapour pressure limits, cavitation sets in, to avoid cavitation (otherwise the efficiency drops and the impeller becomes damaged) the suction head, h_s, is limited so that the pressure at the inlet is equal to the allowable vapour pressure, p_v. Other measures such as minimizing head losses (by choosing large diameters and shorter lengths of suction pipes without regulating valves) are also taken.

The manometric head can be written as:

$$H_m = H_s + h_{fs} + h_{sd} + \frac{V_d^2}{2g}$$

Equation 8-3

Where:

H_m: manometric head
V_s: flow velocity in the suction pipe
V_d: flow velocity in the delivery pipe
H_s: static head
h_{fs}: head losses in the suction pipe (friction, valves, bends etc)
h_{sd}: head losses in delivery pipe (friction, valves, bends etc)
g: acceleration due to gravity

It should be noted that, if the velocity head is negligible, the last term of the above equation is left out in the computation of the manometric head. If the impeller losses are considered, the efficiency (manometric efficiency) of the pump is affected. The total head the pump must develop to overcome the impeller and system losses (in pipelines and pipe fittings) in lifting the water through a given static lift is given by.

$$H = \frac{H_m}{\eta}$$

Equation 8-4

Where:

H	:	total head
H_m	:	manometric head
η	:	manometric efficiency of the pump

8.2.4.2 Practical Suction Head

The maximum suction head (pump setting) can be given by;

$$\boxed{h_s \le \frac{(p_a - p_v)}{\rho g} - \left(h_{fs} + \frac{V_s^2}{2g}\right)}$$

Where

ρ	:	density of water
h_{fs}	:	head losses in the suction pipe (friction, valves, bends etc.)
g	:	acceleration due to gravity
V_s	:	flow velocity in the suction pipe
P_a	:	atmospheric pressure
P_v	:	allowable vapour pressure

The maximum suction lift depends mainly on the altitude (and thus the atmospheric pressure), the water temperature, the intake arrangement and the pump design. In general the maximum suction lifts in Table 8-6 should not be exceeded.

Table 8-6: Maximum Suction Lifts

Altitude above Mean Sea Level (m)	Maximum Suction Lifts (m)
0	5.0
500	4.5
1,000	4.0
1,500	3.5
2,000	3.0
2,500	2.5
3,000	2.0

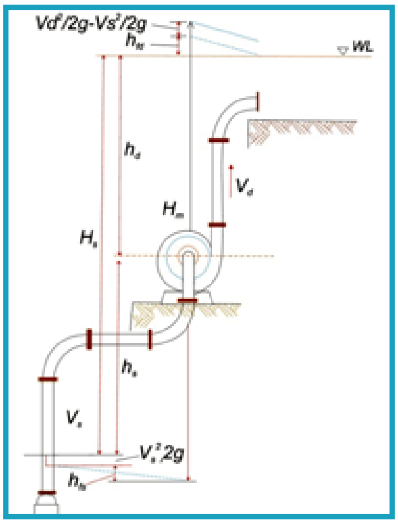


Figure 8-5: Definition Sketch For Pump Installation

Non-compliance with the maximum suction lift restrictions can result in “cavitation” and other associated problems in a pump. “Cavitation” is caused when the atmospheric pressure acting on the water surface in the suction pump is not sufficient to push the water into the pump adequately, to replace the volumes being displaced by the impellers. In such circumstances, the water around the impellers tends to vaporize and form bubbles which explode, forming cavities in the impellers. This phenomenon results in reductions and discontinuities in pump discharges, excessive noise in the pump and reduction in overall performance efficiency.

The phenomenon is symptomatic of insufficient Net Positive Head Suction [NPSH] discussed in more detail elsewhere in this manual. This problem can be remedied by keeping the suction lifts within the permissible limits, adequate design and installation of suction Pipework and operating pumps under the optimal ranges of their characteristics.

8.2.4.3 Net Positive Suction Head (NPSH)

The total head required to push water into the pump, overcome frictional losses as water flows into and within the pump also create sufficient flow velocity corresponding to the required flow rate is called the Net Positive Suction Head (NPSH).The net positive suction head (NPSH) is given by:

$$\frac{(p_s - p_v)}{\rho g} = NPSH = \frac{(p_a - p_v)}{\rho g} - \left(h_s + h_{fs} + \frac{V_s^2}{2g}\right)$$

Equation 8-5

Where:

p_s: pressure at the pump inlet
h_s: suction head
η: manometric efficiency of the pump

The suction specific speed, N_{ss} is given by:

$$N_{ss} = \frac{NQ^{0.5}}{(NPSH)^{0.75}}$$

Equation 8-6

The cavitation number, σ is given by:

$$\sigma = \left(\frac{N_s}{N_{ss}}\right)^{\frac{4}{3}}$$

Equation 8-7

Where:

N_s = the specific speed
N_{ss} = the suction specific speed
σ = the cavitation number.

The suction specific speeds for most centrifugal and axial (propeller) type units range between 4,700 and 6,700. The critical cavitation number σ_c to avoid cavitation according to Novak *et al.* (2007) as suggested (from model tests) can be written as:

$$\sigma_c = 0.103\left(\frac{N_s}{1000}\right)^{\frac{4}{3}}$$

Equation 8-8

The NPSH requirement curve is normally provided by the manufacturers and is related to other physical parameters through the following relationship:

$$\text{NPSH requirement of the pump} \leq B+H_{sta}-H_f$$

Where:

H_{sta} = the static height difference between the centre line of the pump element and the water level on the suction intake side of the pump, in meters. The value is negative if the pump is placed above the water level in the intake, and positive if the pump is placed below the water level in the intake.

H_f = frictional head losses in metres, in the foot valve, the suction pipe and pump casing.

B = a factor which depends on the altitude (and thus the atmospheric pressure), the water temperature and the water vapour pressure of the water as shown in Table 8-7.

Table 8-7: Values of B.

Altitude above Mean Sea Level (m)	Factor B (m)
0	9.4
500	8.9
1,000	8.4
1,500	7.9
2,000	7.3
2,500	6.8
3,000	6.3

When the NPSH requirement of a pump is known (from the NPSH req curve of the pump), the limit of H_{sta} at which the pump should be positioned can be calculated using the relationship given below:

i) $H_{sta} \geq$ NPSH requirement of the pump $+H_f - B$
ii) H_{sta} is negative if the pump is placed above the water level in the intake and positive if the pump is placed below the water level in the intake. Hence, getting a negative result of H_{sta} from the foregoing calculation indicates that the pump can be placed above the water level in the intake, while a positive value shows that the pump must be placed below the water level in the intake.

Some general points about suction conditions are as follows:

i) It is good practice to keep suction pipes as short as is practical.
ii) Suction pipes must be totally airtight. If there are any leaks the pump will be unable to create the vacuum condition for suction to occur.
iii) Suction pipes must be straight and laid to rise continuously to the pump. If there are any leaks in the pipe air pockets will form and the system will become air locked.
iv) Suction pipes must be generously sized, one size larger than the delivery pipe being standard practice. Also all suctions should be fitted with foot valves.

v) Where the distance from the pump mounting point to the water level is greater than the available suction lift either a submersible or a jet pump should be used.

8.2.5 Power Requirements

The power requirements for a water pumping unit can be computed using:

$$P = \frac{Q \times H}{102 \times \eta} \quad kW$$

Equation 8-9

Where:

P = Power required in kW
Q = Pumping rate, l/s
H = Pumping head, in meters (Static head + losses)
η = Pumping efficiency (a value between 0 and 1)

The corresponding energy demand can be calculated using the following formula:

$$E = \frac{Q \times H}{\eta} \quad kWh$$

Equation 8-10

Where:

E = Energy demand, in kWh per year
Q = Pumping quantity of water per day, m^3/day
H = Pumping head, in meters (Static head + losses)
η = Pumping efficiency (a value between 0 and 1)

In practice, the efficiencies of small-capacity pumps are generally low. It can be assumed that the efficiencies are in the range of 30 % for pumps of capacities up to 0.4 kW, and 60 % for pumps of capacities of 4 kW or more.

8.2.6 Estimating Efficiency of a Pump

Careful pump selection and its regular maintenance enable the pump to function as efficiently as possible and consequently lower running cost is obtained. Efficiency of the pump is all about how well the pump converts electrical power to the useful work of moving water. To enhance the efficiency of a pump, the manufacturer's specification of how the pump was designed to operate at is compared with the calculated pump duty for any improvement to lower the pumping cost.

The steps in estimating the pump efficiency include the following:

i) Measure the power consumed, P in kW;
ii) Determine the flow rate, Q in litres per second;
iii) Determine the pressure head and suction lift to obtain the total head, H in meters;
iv) Determine motor efficiency, M_e in percentage: you can assume an efficiency of 85% for motors up to 15 kW, and 90% above 15 kW. Generally, submersible motors are generally 4 points lower than other motors: e.g. for a 22.4 kW 2-pole submersible, motor efficiency is 86%;
v) Determine transmission losses: this loss can be included in terms of drive factor (D_f) e.g. if the loss of energy through the transmission is 5 %, then the drive factor (D_f) is 0.95; for V-belt drives, D_f is 0.9 while for gear drives, D_f is 0.95; and
vi) Calculate pump efficiency, E in percentage:

$$E = \frac{Q \times H}{P \times Me \times Df}$$ **Equation 8-11**

8.3 Power Sources

8.3.1 General

The different types of power sources commonly used for water supply pumps in Uganda include:

i) Diesel engines;
ii) Electric motors powered by the national electric grid;
iii) Electric motors powered by local diesel electric generators;
iv) Electric motors powered by solar power equipment; and
v) Human – powered pumps – hands.

8.3.2 Diesel Engines

The diesel is an internal-combustion engine which burns low-grade fuel oil. The fuel is atomized, forced into the cylinder, and ignited by the compression in the cylinder. Diesel engines are especially adapted to plants of moderate size or where electric current are high. Although the diesel plant can be used to drive any type of pump and has the advantage of being completely self-contained, note should be taken of its high initial cost and requirement of skilled attention. Gearing or any other suitable transmission system to connect the engine to the pump is required.

The effects of altitude and temperature on the power output of diesel engines must be taken into account. Humidity may also have a slight adverse effect on power output.

The following considerations should be undertaken for diesel engines:

i) The decrease in power output can be assumed to be about 01% for every 100 m rise in altitude above mean sea level;
ii) Power output can be assumed to be about 02% for every 0.5 °C rise in air temperature above 30 °C;
iii) A diesel engine driving a water pump, should be able to give 25 – 30% more power than is required by the pump under normal conditions; and
iv) Under normal conditions, diesel engines should run at speeds between 1,500 and 2,000 rev min^{-1}. The engines should be able to be started by hand operation.

8.3.3 Electric Motors

Where available is the cheapest, efficiency is very high and the plant is very compact and is put on or off in a movement. Electric motors are silent in working, free from nuisance of smoke and occupy very little space. When a centrifugal pump is driven by a closely coupled electric motor constructed for submerged operation as a single unit, it is called a submersible pump. The electrical wiring to the submersible motor must be waterproof. The electrical control should be properly grounded to minimize the possibility of shorting and thus damaging the entire unit. The pump and motor assembly are supported by the discharge pipe; therefore, the pipe should be of such size that there is no possibility of breakage.

The fact that motor drive is especially convenient for the centrifugal pump may be a factor for the use of electric power in the water supply pump works. Electric motors should be capable of carrying out the workloads imposed, taking into account the various operating conditions under which the pumps they drive have to work. If the power requirements of a pump exceed the safe operating loads of the electric motor driving the pump, the motor may get damaged or completely burnt out. Attention must also be paid to the operating characteristics of the motor and its supply voltage.

The approximate efficiencies of three-phase motors under full-load are as follows:

i) 70% for 1 kW motors;
ii) 80% for 2kW motors;
iii) 85% for 10 kW motors; and
iv) 90% for 50 kW and larger motors.

It is not practical to choose a motor of a capacity less than 0.25 kW, even if the actual power requirement is smaller. In order to avoid overloading an electric motor, the rated power of the motor should exceed the power requirement of the pump by the following percentages:

i) 50% for pumps requiring up to 1.5 kW;
ii) 30% for pumps requiring between 1.5 and 4 kW;
iii) 20% for pumps requiring between 4 and 8 kW;
iv) 15% for pumps requiring between 8 and 15 kW; and
v) 10% for pumps requiring over 15 kW.

8.3.4 Solar Power

Solar-powered water pumps or photovoltaic pumps (PVP) are an effective alternative to conventional gas or electric pumps. Modern pumps are powered by solar energy effectively and used in different parts of the world. PVP systems offer numerous advantages over water supply systems utilizing conventional power:

i) PVP systems may be the only practical water supply solution in many regions where the logistics make it too expensive or even impossible to supply diesel generators with the required fuel;
ii) PVP systems are ideal for meeting water requirements for villages between 500 and 2,000 inhabitants and small scale irrigation purposes (up to 3 hectares);
iii) PVP systems run automatically, require little maintenance and few repairs;
iv) In areas where PVPs have entered into competition with diesel-driven pumps, their comparatively high initial cost is offset by the achieved savings on fuel and reduced maintenance expenditures;
v) The use of solar energy eliminates emissions and fuel spills thereby making photovoltaic pumps an environmentally sound and resource-conserving technology;
vi) There is no need for complicated wiring for the electricity and outside fuel is not needed;
vii) They do not require batteries, which are expensive and need a lot of maintenance; and
viii) The maintenance of a PVP system is restricted to regular cleaning of the solar modules.

Depending on the water quality, the only moving part of the system, the submersible motor pump, has to be checked every 3 to 5 years.

Taken together, these reasons are an incentive for Ugandan water authorities as well as private investors to decide in favour of a PVP system as against conventional pumping techniques.

The three components of a solar-powered water pumping system are:

i) Photovoltaic (PV) array (solar cells);
ii) Electric motor; and
iii) Water pump.

The PV array generates direct current (DC) electricity when exposed to sunlight. This electricity is fed into the electric motor and which in turn drives the water pump. Optional components of the system include the following:

i) Controllers (for regulating current and/or voltage);
ii) Inverters (for converting DC power from the PV array to alternating current (AC) power for certain types of motors);
iii) Electronic maximum power tracking devices, MPPT (to obtain a more efficient operation of the array and the motor); and
iv) Batteries (used for both voltage regulation and energy storage and also for generating the required starting currents needed to overcome high electric motor starting torques).

The selection of PV arrays and associated equipment should always be made in consultation with the relevant manufacturers. The operating principle behind any photovoltaic pumping system is quite simple. A solar generator provides electricity for driving a submersible electric pump, which in turn pumps water into an elevated water tank that bridges night-time periods and cloudy days. Force of gravity causes the water to flow from the tank to public water taps and watering points for livestock or to the irrigation system. On a clear, sunny day, a medium-size PVP system with an installed power of 2 kW will pump approximately 35 m^3 of water per day to a head of 30 meters. That amount of water is sufficient for communities with populations up to 1,400. Additional performance data for the various system designs are indicated in Figure 8-6. Today's generation of PVP systems is highly reliable. For the most costly part, the PV generator, the manufacturers give a 20 year guarantee on the power output. A crucial prerequisite for the reliability and economic efficiency is that the system be sized appropriate to the local situation.

The power output of a PV array is directly proportional to the solar irradiation falling onto the array. The power flow through a typical solar-powered water pumping system is in Figure 8-6. Accordingly, the best efficiencies that are expected are as follows:

i) PV array — 11% of the total solar power received by the PV array; and
ii) Motor-pump unit — 4.5% of the total power received by the PV array

The formula below can be used to estimate the daily solar power requirement of a water pumping system, in kWh.

$$P = \frac{\rho \times g \times Q \times H}{3600 \times e} \quad kW$$

Equation 8-12

Where:

P = Power required in watt-hours/day
g = acceleration due to gravity = 9.8 m/s^2
ρ = density of water = 1000 Kg/m3
Q = daily water requirement, m^3/d
H = Total Pumping head, in meters
e = Overall mechanical efficiency of the system.

In practice, the overall mechanical efficiency of the system is about 30%.A typical performance curve, showing the power generated by a PV panel during the day is given in Figure 8-7. Solar panels are specified by the Peak Power Rating, the power output of the PV panel in watts when the panel is receiving radiation of 1,000 watts per square metre, at ambient temperature of 25 °C. The available power in watt-hours per day is equal to the hatched area of Figure 8-7. This is area is equal to the Peak Power Rating of the panel multiplied by 5 hours. Thus, if a panel is rated at 50 Watts Peak Power, then the useful Daily Output is 250 Watt-hours.

Thus the number of PV panels required can be found using the formula below:

$$\text{Number of panels required} = \frac{\text{Daily Power Required}}{\text{Peak Power Rating} \times 5} \qquad \textbf{Equation 8-13}$$

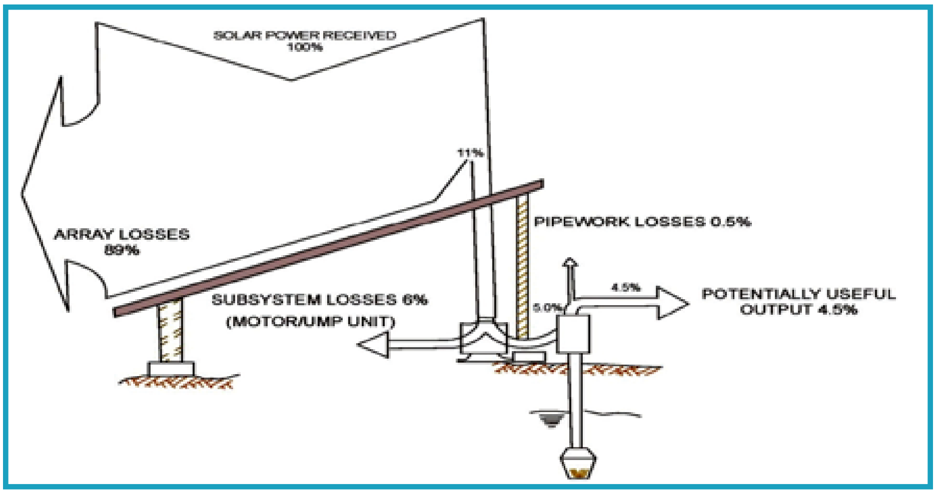


Figure 8-6: Losses in a Typical Solar PV Pumping System.

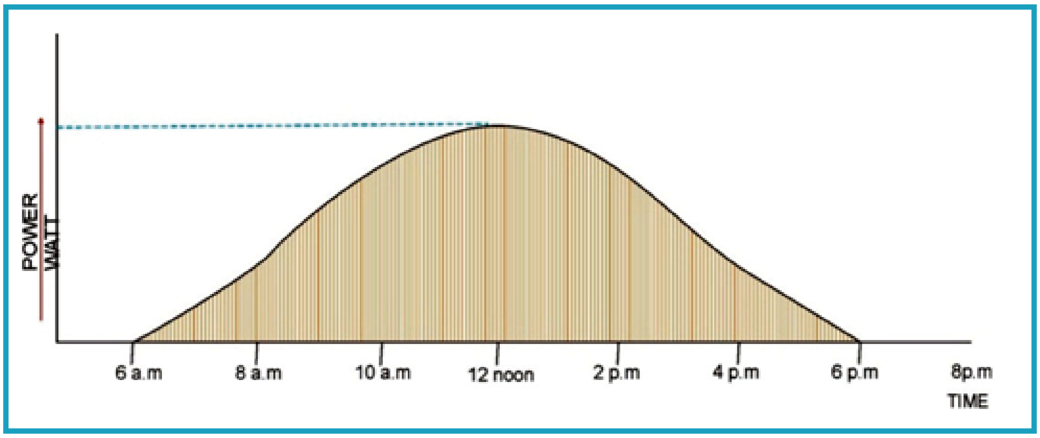


Figure 8-7: Diurnal Power Generation from a Solar PV Panel.

When designing solar-powered water pumping systems, keep in mind that:

i) PV arrays panels are prone to vandalism and thus they should be protected by fencing; and
ii) The shading of PV arrays should be avoided by siting obstructions on the south side of the arrays and by keeping the surrounding vegetation always cut.

8.3.5 Sizing a PVP-System

The PVP system is sized on the basis of the findings from a local data survey. While an onsite survey of meteorological and climatic data would be worthwhile it is usually hindered by lack of time and money. Many systems are based on the known data of a nearby reference location for which relevant measured values are available.

If it is possible to visit the intended location, the following field data should be gathered:

i) Water quality;
ii) Demand for water in the supply area;
iii) Pumping head with allowance for friction losses and well dynamics; and
iv) Geographical peculiarities, e.g., valley locus.

It is also important to include sociological factors in the planning process. The future users should be involved in the data-gathering process at the intended PVP site in order to make early allowance for their customs and traditions in relation to water. Women in particular must be intensively involved in the planning, because they are the ones who are usually responsible for maintaining hygiene and fetching water. Thus, the planning base for each different location should cover both technical and sociological aspects.

The technical planner can choose from a number of design methods of various qualities. The most commonly employed approaches are outlined below.

8.3.6 Estimation of PV Generator Output

To arrive at a first estimate of how much the planned PVP system will cost for a just selected site, it is a good idea to first estimate the requisite size of the PV generator. This, however, presumes knowledge of the essential sizing data, namely the daily water requirement within the area of supply (V_d), the pumping head to be overcome by the pump (H), and the mean daily total of global irradiation (G_d) for the design month.

A simple arithmetic formula allowing for the individual system component efficiencies can be used to calculate the required solar generating power, P_{SG}. The equation reads:

$$P_{SG} = 11.6 \times \frac{H \times V_d}{G_d}$$

Equation 8-14

According to this equation, it takes a 3.5-kWh PV generator to deliver water at the rate of 30 m³/d at a head of 50 m for a daily total global irradiation of 5 kWh/(m²*d). This gives the planner an instrument for estimating the size of the PV generator and, hence, the cost of the planned system at the time of site selection.

There are three categories of PVP products on the market:

i) Solar Products manufacturers;
ii) Pumping products manufacturers; and
iii) System integrators.

Whatever source of equipment supplier is selected, installation must be done in accordance with the suppliers manuals and conditions and the relevant regulations and any case relevant qualified personnel must be consulted at all times.

8.3.7 Wind Energy

For adequate usage of wind power, the wind speed should be higher than 2.5 to 3 m/s for at least 60% of the time. The windmill should be placed above surrounding obstructions such as trees or buildings within 125 m; preferably, the windmill should be set out on a tower of 4.5 to 6 m high. Consider protection by provision of an automatic lubrication system or covering of the windmill driving mechanism.

In order to directly pump water by a windmill, there is need to match the characteristics of the local wind regime, the windmill and the pump. This therefore means that, the manufacturers should always be consulted regarding the selection of the equipment.

The discharge Q that can be pumped by a windmill can be estimated by the formula below:

$$Q = \left(\frac{2.8 \times D^2 \times V^3 \times e}{H} \right) \; l/min$$

Equation 8-15

Where:

D = wind rotor diameter in meters,
V = wind velocity in meters per second,
H = pumping head in meter,
e = wind to water mechanical efficiency, value 0-1.

Windmills with rotor diameter between approximately 2 m to 6 m are usually available. The efficiency, e will rarely exceed 30%.

The power in the wind is proportional to the wind speed cubed:

$$P = \frac{1}{2} dAV^3$$

Equation 8-16

Where:

P = power (KU)
d = density of air (1.2 kg/m^3)-ASL
V= instantaneous free steam wind velocity (m/s)

About 5 years of data would give the designer reasonably representative averages following the fact that the monthly wind speed varies greatly between 10 % to 20 %. Therefore, the relationship from the preceding formula of power availability is extremely sensitive to wind speed e.g. doubling the wind speed increases the power by 2^3 or more.

In sizing of windmill, the diameters of their wind wheels are used. The larger the diameter of the wind wheel, the greater the elevation to which the water can be pumped. If the windmill is at the top of a hill, a reduced tower height can be used. Table 8-8 below shows the desirable minimum heights to which the windmill towers for average conditions as function of the windmill diameter.

Table 8-8: Sizing of Windmill.

Wind Wheel Diameter (m)	Windmill Tower Height (m)
1.8	7.5
2.4	9.0
3.0	9.0
3.6	12.0
4.2	12.0
5.1	13.5
6.3	13.5
7.5	16.5

For successful operation of a wind pump, at least wind speed of 2 to 3 m/s is required. Effort should be made to acquire a wind map of the area to guide with the wind speed throughout the year. This information is available from the Meteorological Department in Entebbe; however, it may require interpretation and organisation to ensure that it is applicable to the area in question.

8.3.8 Pump Electrical Control

Most pumps are powered by electric motors and a correct electrical installation is essential to ensure effective operation of the pump.

8.3.9 Motor Starting

The type of motor starter depends upon the type of motor being installed.

8.3.9.1 Small Single Phase Motors

Most small single-phase pumps are designed to operate without a remote starter and can be directly connected to the mains via an appropriate fuse or MCB. These pumps have built in thermal overload protection, which stops the motor in the event of an electrical or mechanical overload.

8.3.9.2 Large Single Phase Motors

Large single – phase motors, usually greater than 1.5 HP usually do not have built in motor protection and a direct-on-line starter should be used. If in doubt, contact the pump supplier.

8.3.9.3 Small Three-Phase Motors

Three phase motors for centrifugal surface pumps up to 7.5 HP need a direct-on-line starter with appropriate overload relay.

8.3.9.4 Large Three-Phase Motors

Three phase motors from 7.5 kW to 30 kW are usually specified with a Star/Delta starter with appropriate overload relay.

i) Borehole Pumps - Due to the particular design of a borehole pump, motor manufactures recommend the use of Direct-on-line Starters for all motor sizes up to 25kW and Auto-Transformer or suitable alternatives for motors above that size. Star/Delta starters are not recommended for boreholes because the 3 phase motor starts with low current gradually building its speed to the point where delta connection takes over. This is disadvantageous especially in cases of low torque causing the pump motor to stall and also the motor can stall in transition. This could cause a return of water already pumped causing a water hammer on the pump, drop in water column and introduction of air pockects in the rising main. Therefore, soft starters are a better option although costly; and
ii) Motor Protection - Current overload can be caused by electrical or mechanical overloads in the motor or poor quality electrical supply voltages. All motor starters provide protection against current overloads, the protection level being determined by the overload relay setting. It is very important to ensure that the overload relay fitted to the starter is correctly rated and set, these being determined by the full load current of the motor. The correct overload setting is 5% above the full load motor current for DOL starters and 60% of the full load current for star/Delta starters.

Irrespective of whether a starter is fitted or not, all pump installations must be provided with a switched current protective device (either a fuse or MCB) which should be rated approximately 50% above the full load motor current.

For more sophisticated or high cost installations additional protection is often necessary and protective relays for sensing over and under supply voltage, phase asymmetry and phase failure are often provided. These units are installed as a supplement to the motor starter. Also available are electronic controllers which as well as providing a normal starting facility (usually DOL), also provide integrated current, voltage and run dry protection. It is important to discuss options with the pump supplier so the best motor protection accessories are specified.

8.4 Design of Sump and Pumping Mains

8.4.1 System Characteristic, Duty Point, and Operational Range

The efficient design of a pumping station largely depends on the piping system used to convey the fluid. Friction and other losses in the system, which are a function of discharge, have to be overcome by a pump, the performance of which is interrelated with the external pipe (system) characteristic. The pump characteristic at a given speed, N, is a function of discharge and is written as

$$H_m = AQ^2 + BQ + C \qquad \textbf{Equation 8-17}$$

Where A, B and C are the coefficients which can be evaluated from its Q-H curve. The system curve is given by:

$$H_m = H_s + h_m + h_f \qquad \textbf{Equation 8-18}$$

Where h_m = minor system losses
h_f = major system losses
H_m = total (manometric) head

The minor and major losses can be expressed as KQ^2, K being the appropriate loss coefficient. The solution of the two equations above, either analytically or graphically (see Figure 8-8), gives the duty point of the installation at which the pump delivers the required discharge. It is important that the duty point coincides with the peak efficiency of the pump for its economical operation. For varying discharges, the unit may be throttled over an operational range at the expense of the head. However, the extent of the operational range may have to be limited to one giving reasonably high pump efficiencies.

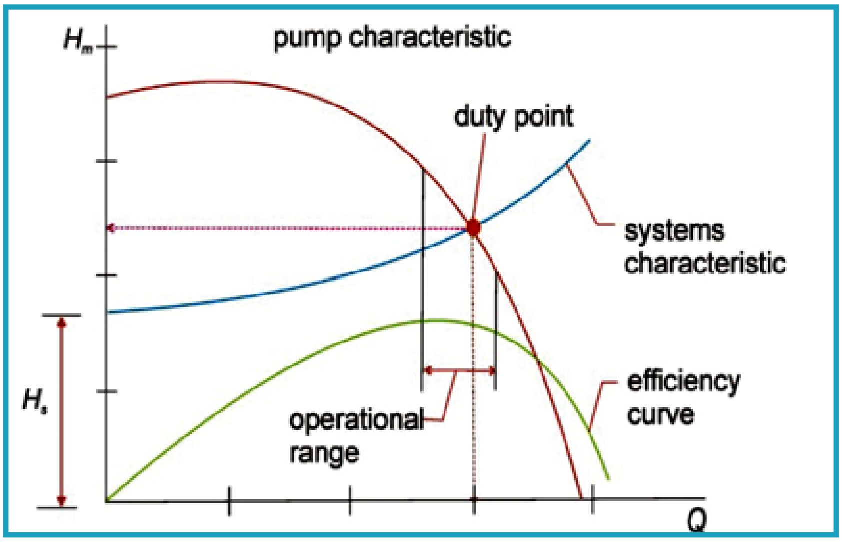


Figure 8 8: Pump-Line Characteristics

8.4.2 Selection of Pipeline Diameter

The best possible diameter of a pipeline system for a pumping unit depends on the system characteristic. The various pipeline systems which could be considered as matching the pump characteristic are shown in Figure 8-9. It must be noted that, each operational point corresponds to a particular efficiency of the pump, and the system selection largely depends on the discharge-head requirements and on pump efficiency.

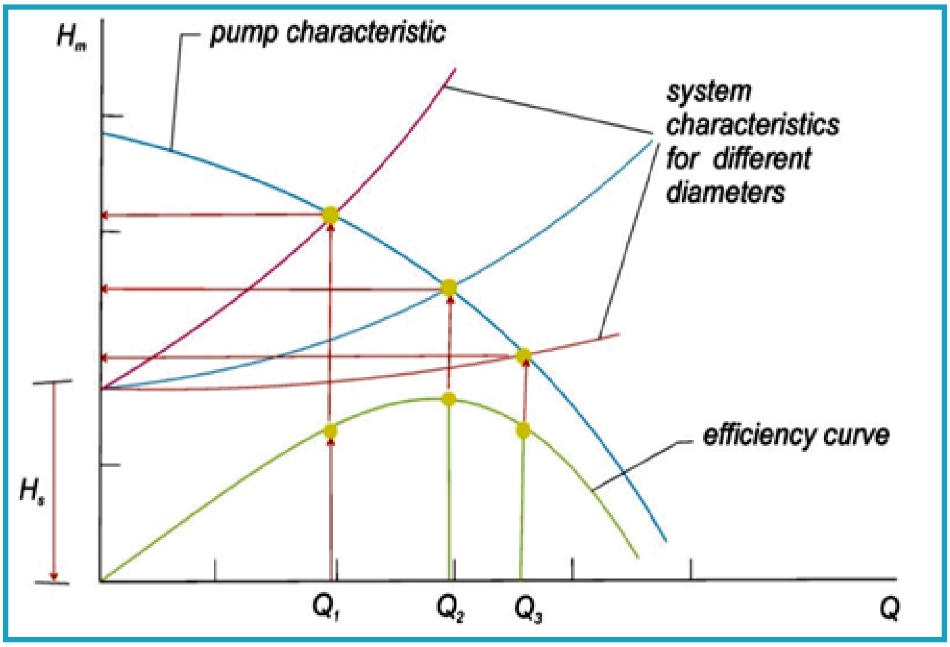


Figure 8-9: Selection Of System Pipeline Diameter

8.4.3 Sump Design

The most important aspect of pumping installation is the suction (sump), with a good intake (inlet to the suction pipe) arrangement. In the case of shallow submergence of the inlet, the formation of a local vortex at the water surface is often observed. Such a situation may also arise if he volume of water in the sump is inadequate, in which case the sudden starting of the pump lowers the water surface. Equally sudden steep velocity gradients, high velocities, sudden changes in the flow direction as water enters the bell-mouth inlet in the sump and several pump units set in a long narrow sump with inflow from one end also aggravate the formation of vortices.

The vortex entrains large quantities of air into the pump suction pipe. This causes a drop in efficiency and structural vibrations, and increases the corrosion damage of the pump and its accessories. A good sump design must therefore avoid the formation of vortices; this can be done by directing the flow uniformly across the sump width with an approach velocity of around 0.3 m/s and by avoiding abrupt expansion of the side walls and large stagnation zones in the sump (minimizing large-scale swirling flow). For larger expansions ratios {[area at outlet/area at inlet]>2} vanes may have to be provided for uniform flow distribution. The pump intake must be located in the direction of the approach flow if possible. Multiple intakes from the same sump should be separated by dividing walls (to minimize interference). The most effective free-surface vortex-suppressing device is a horizontal floor grating, installed about 100 to 150 mm below the water level. Other devices such as floating rafts, grating cages, and curtain walls are also used as vortex suppressors. Provision of a bell-mouth entry at the inlet to the suction pipe minimizes entrance losses and facilitates smooth axial flow.

For intakes with proper approach flow conditions (without any vortex-suppression devices) the minimum required submergence could be written as:

$$\frac{h}{d} = a + bF_{rd}$$

Equation 8-19

$$F_{rd} = \left(\frac{V}{gd}\right)^{\frac{1}{2}}$$

Equation 8-20

Where:

h : depth [m] of submergence to the centre of intake pipe of diameter, d [m]
F_{rd} : Froude Number, [-]
V : velocity in the intake pipe, m/s
a, b : constants

The constant *a*, varies from 0.5 to 1.5 while *b* varies from 2 to 2.5 (Knauss, 1987). At low F_{rd} ($\leq$ 0.3, i.e. large-size intakes) a value of *a*, of at least 1 is recommended.

The minimum sump volume for good flow conditions also depends on the maximum allowable number of pump starts in a given time. Start-up of an electric motor generates considerable heat energy in the motor, and hence the number of starts must be limited. The minimum sump volume, V_{min}, between stop and start of a single pump unit is given by:

$$V_{min} = \left(\frac{Q_P T}{4}\right)$$

Equation 8-21

Where:

Q_P *:* the pumping rate, *T*: Time between starts.

The equation above suggests that for a frequency of 10 starts per hour (T = 6 min), the minimum volume is 1.5 times the pumping rate per minute. When two or more pumps are used, the start levels are normally staggered and the equation is applied to the largest pump.

For a multi-system, an additional volume of 0.15 times the plan area of the sump (in m^3) should be provided.

8.4.4 General Design Considerations of Pumping Stations and Mains

Rising mains (pipe line systems) from a pumping station normally follow the ground contours, with their carrying capacities dependent on their hydraulic gradients. The gradient of a rising main is given by the pressure and friction heads determined by the characteristics of the pump-pipeline characteristics. When boosting the flow through an existing pipeline by installing a booster pump the elimination of suction troubles may have to be considered.

Booster pumps are normally commissioned whenever there is an increased demand (e.g. peak day time consumption). Automatic cut-in and cut-out arrangements of the booster must be provided so that the pipeline is not subjected to high pressures during off-peak (night-time) periods when the demand reduces considerably. A simple and economical device to control the operation of the booster pump is to provide a balancing tank in the system at an appropriate point with the water levels in the tank triggering the pump on and off according to demand (see Twort *et al*., 2000).

Pumping stations for water supply from river intakes or boreholes are normally designed to discharge continuously over a period of 20 – 22 hours a day. Booster may also need storage facilities. Pumping mains are usually designed for velocities of around 0.9 – 1 m/s when supplying water at a constant rate throughout the day. This may be doubled for short-period pumping. Prior to the final selection of the pipeline diameter, economical analyses balancing the costs of large diameter pipelines and their maintenance, and the savings in pumping costs due to the reduction in friction losses, must be carried out.

8.4.5 Multiple Pump Design Considerations

8.4.5.1 Introduction

A water system may include multiple pumps designed and operated to provide variable design flows to its service area. The configuration of a multiple pump operation can be "parallel" or in "series." The design engineer should be familiar with the procedures for sizing multiple pumps under either the parallel or series operational modes.

Pump combination is one of the important factors considered in the sizing of pumps and pumping stations. The idea behind pump combination is to use series of smaller pumps rather than one larger unit as an economic alternative. This allows a smaller pump operating at a higher efficiency to be used during low demand periods with resultant saving in running costs. However, what needs to be satisfactorily resolved by the designer to ensure feasibility of a multiple pump installation is addressing the fact that the change from single pumps to multiple operations may affect individual pump efficiencies and maintenance costs.

8.4.5.2 Parallel Pump Operation

The combined pump head-capacity curve is determined at the same head. Just add the capacities of the individual pump curves after they are modified to account for ordinary friction losses that occur as part of the system head-capacity curve. The point where the combined curve and the system head-capacity curve intersect yields the total capacity of the combined pumps and the modified head at which each operates. The actual total capacity is normally less than the sum of the individual capacities of each pump.

8.4.5.3 Series Pump Operation

The combined head-capacity curve is determined by adding the head of each pump at the same capacity (pumping rate). This mode is used to increase the head capacity of the pumping station. The combined operating head will be greater than each individual pump can provide, but not as great as their sum. Several textbooks cover the design and selection of pumps for multiple pump operations (Heald 2002; Sanks *et al*. 1998; Karassick 2001; White 1998; Lobanoff and Ross 1992; Hicks and Edwards 1971).

8.5 Pump and Pipe Installations in the Pump House

8.5.1 General

Pump installations with delivery heads exceeding 150 M should be avoided. Such high pressures put considerable strain on pumps and pump pipe work systems and they are likely to cause considerable operation and maintenance problems. Surface pumps, electric motors, diesel engines and diesel electric generators should be placed on concrete pedestals, especially designed and isolated from the rest of the floor slabs of the pump houses.

8.5.2 Installation of Pump Controls

8.5.2.1 Introduction

Level controllers can be used to start and stop pumps either at high level or low level. A conventional installation is shown in Figure 8-10.

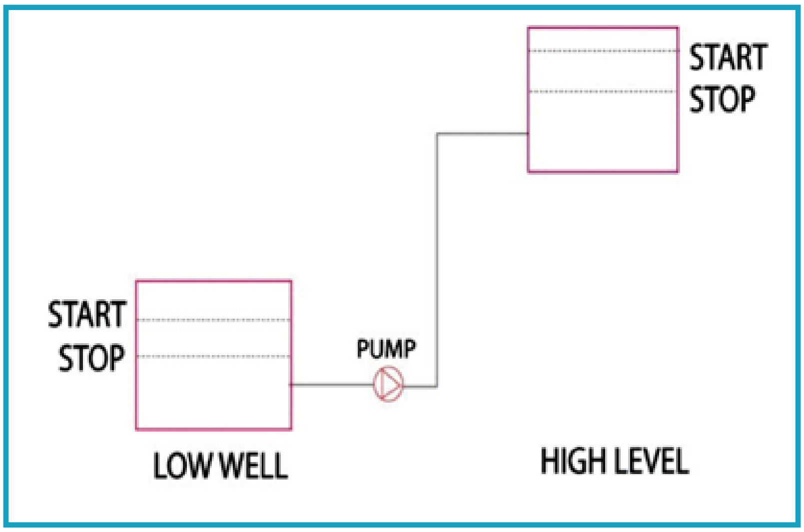


Figure 8-10: Typical Pump Control Installation

8.5.2.2 Low Level

Control is provided for pump protection to stop the pump in the event that the supply reservoir empties.

8.5.2.3 High Level

Control is provided to stop and start the pump according to demand at the delivery reservoir. Three types of level controllers are popularly used:

i) **Float Switch -** This is the simplest device and operates via a float activating lever which makes and breaks the electrical circuit. High and low level switching is adjusted by stops on a string. The advantage of these devices is their low cost, though they tend to be the list reliable of the options available.

ii) **Paddle Switch -** The paddle switch is suspended above water level and makes and breaks the electrical circuit by changing from the vertical to the horizontal position. The device is reliable, economic and simple to install, though is restricted to fairly small level differentials.

iii) **Electrode Control -** This is an electric device operated by means of suspended electrodes. Though the most expensive option, it is easy to install in difficult sites and provides precise level control.

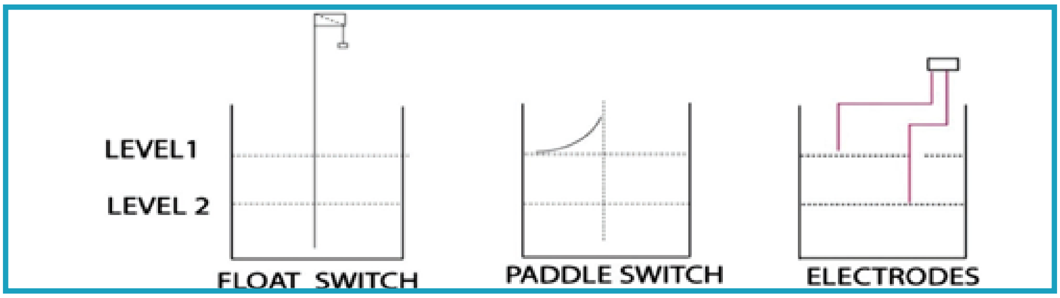


Figure 8-11: Types of Level Controllers

Level controllers are generally operated by making or breaking the control circuit on a relay (often incorporated into the motor starter), which starts and stops the pump motor. For small pump, switches can be installed directly onto the pump's electrical supply, though care should be taken to make necessary connections on the neutral line to prevent switch damage.

8.5.3 Pressure Control

Pressure switch devices are used for two functions:

i) **Pressure Control**: - To start and stop a pump instead of a level controller. Such installations are necessary when the pump is some distance from the delivery point and cabling would be expensive. Pressure control requires an appropriately rated pressure switch wired through the pump starter to start and stop the pump at pre-set pressures. Pump cycling is avoided by using a time relay on the starter and a small surge vessel.

ii) **Pressure Supply**: - To control the operation of one or more pumps depending upon site demand. For larger systems a pressure switch is used to switch the pump. Pump cycling is controlled by a larger air vessel and a time delay relay should not be fitted. For smaller systems specialized integrated control devices are available including the 'Presscontrol' and 'Hydrascan' units which switch off at low flows and switch on at low pressures. System design and vessel sizing should be referred to a Pump Specialist.

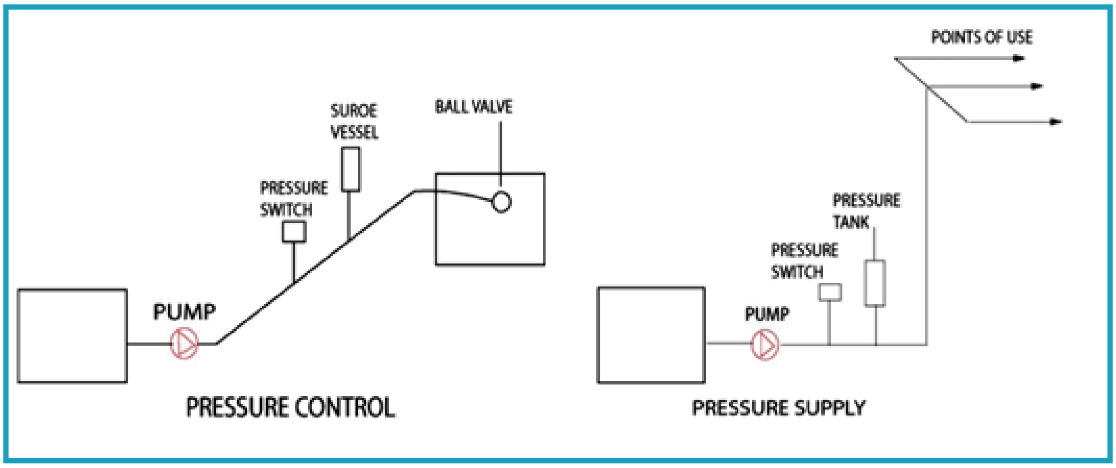


Figure 8-12: Types Of Pressure Control Installations

8.5.4 Electronic Controllers

8.5.4.1 Introduction

Multi-function electronic controllers are the modern and most effective form of pump control which offer the benefits of much simplified installations and a wide variety of control features. Various types are available as follows:

8.5.4.2 Press Control

Activates pump starting when line pressure drops to a pre-set level and stops in a low condition, which also provides protection against dry running when there is no water in the suction.

8.5.4.3 Easypress

Offers similar control features to the presscontrol, though in addition includes auto restart in the run dry condition when water returns and adjustable cut-in pressures. The units include a built-in pressure gauge.

8.5.4.4 Brio

Activates pump starting when the line pressure drops to a set, though adjustable line pressure and stops on adjustable pressure so the stop pressure is not the closed head of the pump. The unit also self-adjusts to pressure drops resulting from system water seepage reducing pump cycling frequency.

8.5.4.5 Torrium

The most sophisticated controller available that features an adjustable start pressure, pump stopping on low flow, an intelligent processor that adjusts pump cycling to system conditions and also protects against high running current and low incoming power supply voltage.

8.5.4.6 Variable Speed Dives

Controls pump output to a fixed adjustable line pressure by varying the pump motor speed. Controllers also provide protection against running dry and non-standard incoming power supply voltage.

8.5.5 Electrical Installation

8.5.5.1 Introduction

Correct electrical installation for all pumps is essential to ensure adequate safety and operational efficiency. The following must be considered:

8.5.5.2 Control Panels

A properly specified control panel is vital for the long life and efficient operation of all pumps with various options being available.

8.5.5.3 Cable Size

The appropriate sized cables must be used to avoid excessive voltage drops over the cable length. Voltage drops should be less than 5% from the power source to the pump, this being calculated by use of the voltage drop.

$$\text{Voltage drop} \geq L\ x\ A\ x\ V_d$$

Where $L \geq$ Cable length (m)

$A \geq$ Pump operating current (Amperes)

$V_d \geq$ Voltage Drop Amp/m for the cable size selected.

8.5.5.4 Earthing

All installations must be correctly earthed either to the mains or a separate earth rod installed adjacent to the control panel. Earthing has very important safety implications and a qualified electrician must be consulted on this aspect of an installation.

8.5.5.5 Wiring

Use only professional panels correctly wired to the laid down electrical standards.
Inadequately wired control panels can be very dangerous and also lead to pump problems caused by bad connections.

8.5.6 Pump Pipe work

Where pumps are installed to operate under suction lift conditions, as discussed elsewhere in this manual, separate suction pipes should be provided for each pump. In such circumstances, the suction pipes should always be equipped with strainers and non-return foot (non-return) valves.

A sluice valve and a non-return (check) valve should also always be provided on the discharge pipe of a pump. Pumps must be provided with appropriate pressure gauges in the suction and discharge pipes. A by-pass pipe should be provided between the discharge pipe and the suction pipe, to facilitate the cleaning of the strainer by flushing. Pipework close to a pump should be supported so as to avoid vibrations being transmitted directly to or from the pump.

8.5.7 Head Losses in Pump Pipework

Figure 8-13 show head losses in various configurations of pipework. The nomograph is useful for a variety of fixtures shown in Figure 8-14.

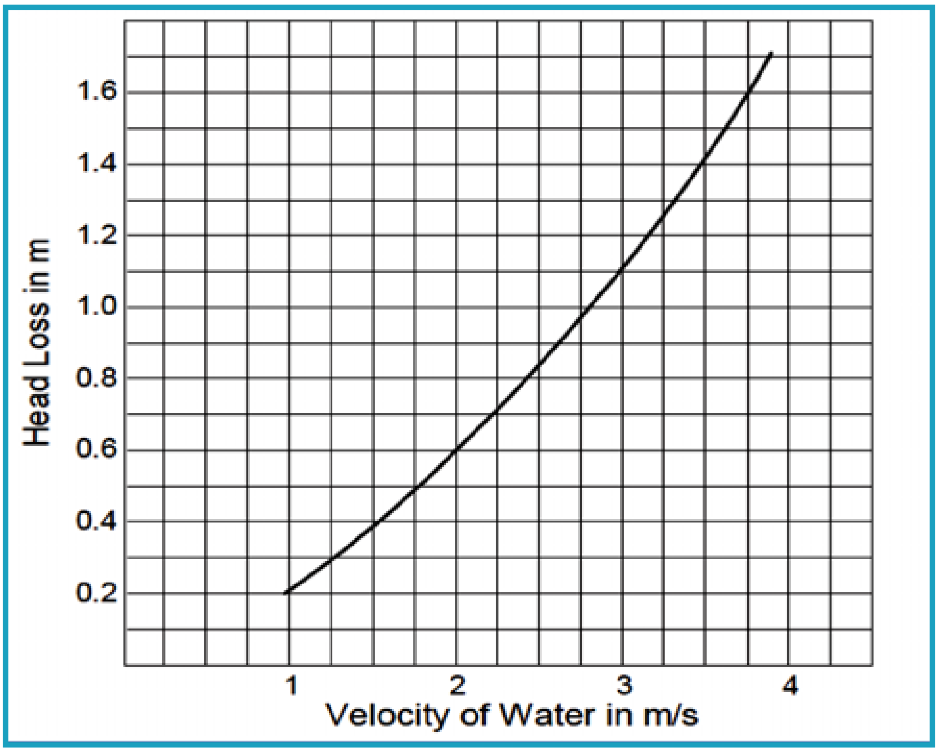


Figure 8-13 Head Losses in Pump Pipework

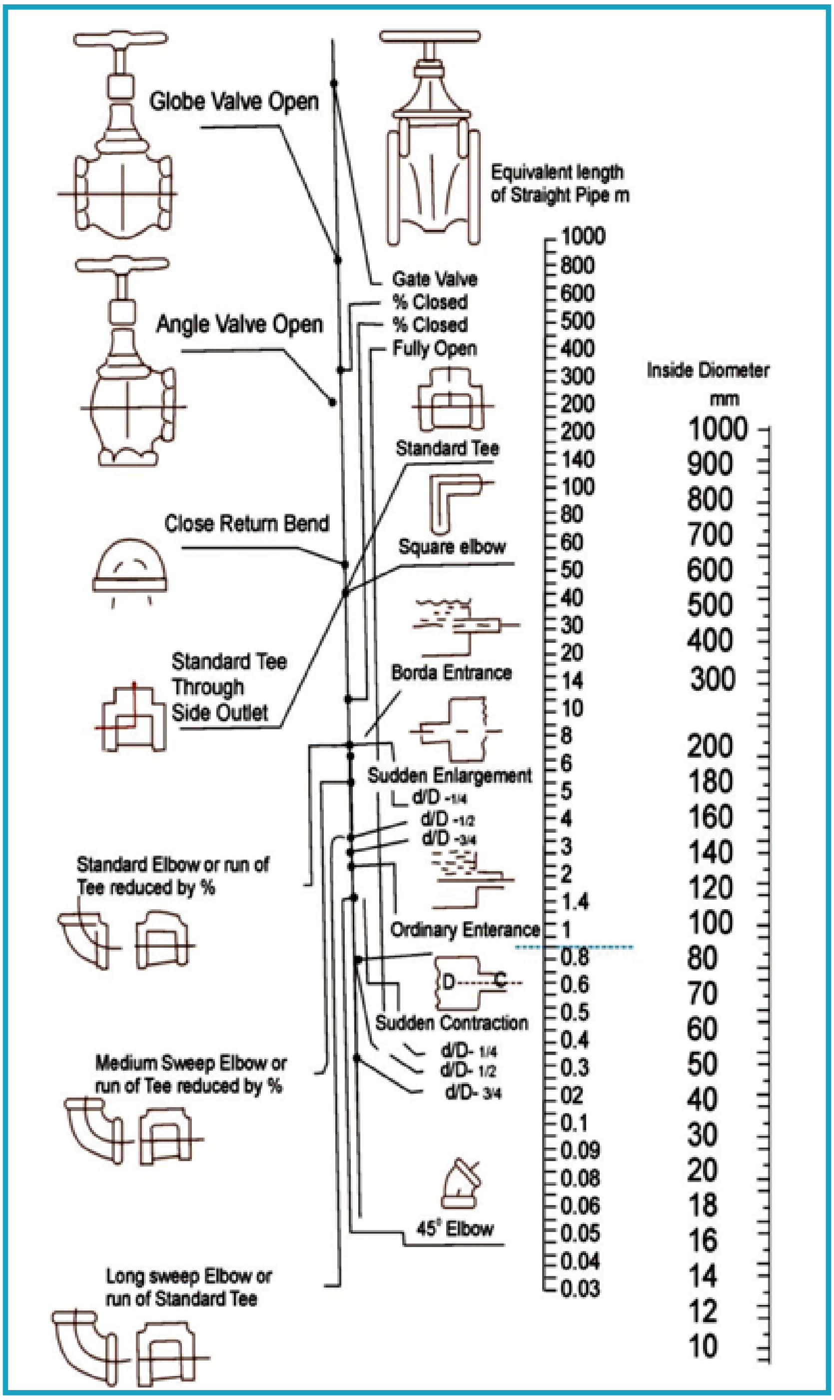


Figure 8-14: Head Losses in Pipe Fittings.

8.5.8 Pump Suctions

The design and installation of pump suctions is a critical factor in achieving the required Net Positive Suction Heads (NPSHreq) of the pumps. Suction should be designed in such a way as to minimize suction losses. Suction pipes should always be as short as possible, slope uniformly upwards towards the pump and be without upward loops. Where looping is unavoidable, a downward loop should be used as illustrated in Figure 8-15. Particular care must be taken to ensure uniform water entry into double-suction pumps, so that one side is not left 'starving'. 'Starving' reduces pump capacity and can cause over-loading. To stop air from being drawn into a suction pipe, the pipe must be completely air-tight and its inlet end should be submerged by a water depth of not less than 1.0 m. It is recommended to use suction pipes with bell-mouth inlet ends combined with foot valves and a strainer. The velocity of water through the pump suction should not exceed 10 m/s.

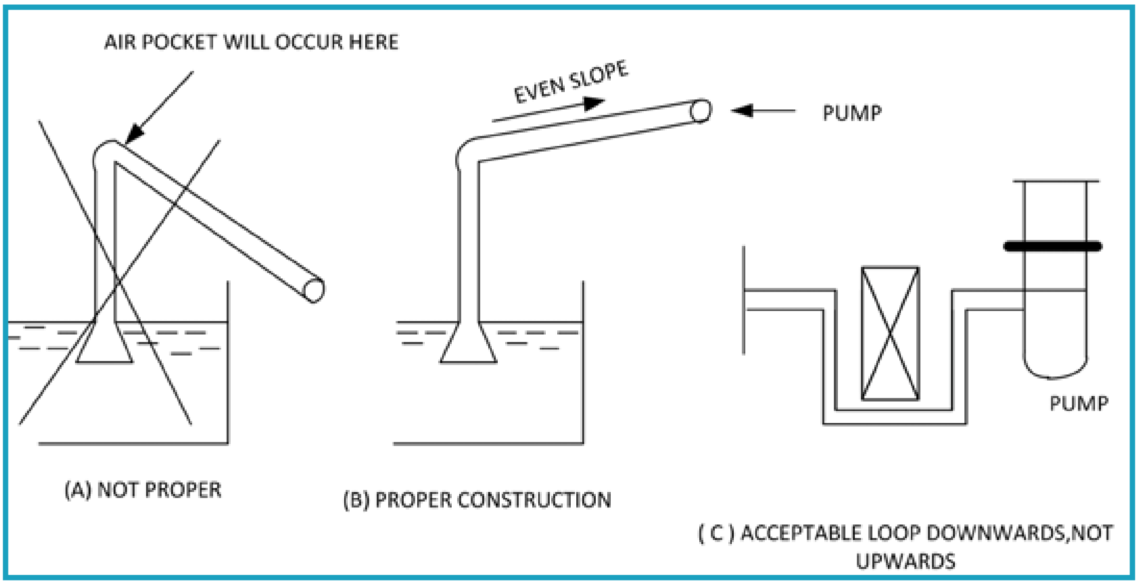


Figure 8-15: Pump Suction Design

(A) Not proper (B) Proper construction (C) Acceptable loop downwards, not upwards.

8.5.9 Spare Parts

It is recommended that at the least 1 year's supply of the spare parts and tools recommended by the suppliers of the pumping equipment to be installed, should be included in the design of the pump installation.

8.6 Operation Time

8.6.1 General

Generally, the optimal number of pumping hours per day should be determined after carrying out appropriate technical and economic analyses. The analyses should take into consideration of the capital costs of the pumps, pipelines and storage reservoirs, and also the recurrent costs of electric power or other energy for running the pumps.

8.6.2 Raw Water Pumping

Raw water pumping stations should normally be designed for 24-hour operation, to match treatment works which are similarly recommended to be designed for 24-hour operation. It should be noted that designing a pumping station for 24-hour operation does not necessarily mean that individual pumps will be running continuously. The pumps should be run alternatively.

8.6.3 Borehole Pumps

A borehole pump should normally work for 24 hours a day as this, for a given yield, reduces the required pump speed and/or pump size, and it usually results in optimal utilization of the borehole. However, when the water demand is still much lower than the capacity of the borehole installed, then the pump may need to be run only for a few hours.

8.7 Standby Units

8.7.1 System Reliability

The stand-by provisions recommended in this section should be considered in conjunction with the other precautions which have to be taken to safeguard the reliability of the water supply system, such as emergency storage provisions discussed in Chapter 9 – “Treated Water Storage”.

8.7.2 Raw Water and Treated Water Pumps

A pumping station should have one stand-by pump with the same capacity as each of the duty pumps. Hence, for a station designed for one duty pump, a second similar pump should be installed as a stand-by unit. For a station designed for two similar duty pumps running in parallel, a third similar pump should be installed as a stand-by unit.

8.7.3 Borehole Pumps

Normally, boreholes equipped with electric motor-driven or diesel engine-drive pumps are not to be provided with stand-by units.

8.7.4 Electricity Supply

In rural areas, where a national electric power supply grid exists, reserve power supply sources such as electric generators and diesel engines should not be provided. Owing to the continuing unreliability of the national electric grid power supply in Uganda, it is recommended that for major water supply schemes especially; in large towns and urban centres local electric generators or stand-by diesel engine-driven pumps be provided to deliver at least 50% of the normal water supply, during periods of national electric grid power outage.

8.8 Phasing

8.8.1 Design Period

Pump houses and pump pipework installations should be designed for the ‘Ultimate’ year requirements. However, pumps and their power sources should normally be designed for periods not exceeding their economic life, which is normally 10 years.

8.8.2 Pump Units

Water demand and thus the pumping requirement at the beginning of the design period of a water supply scheme, are usually considerably less than the requirements at the end of the design period. This disparity should be allowed for by adding pump units in pace with the increase in water demand, or by appropriately increasing the speeds of belt-driven pumps as the requirements increase.

Due regard should be paid to the fact that the actual operating duty point of a pump will shift away from its designed operating duty point, if the flow in the pumping main changes or seep of the pump is altered from the original design assumptions. The pump and pumping main characteristics should therefore be studied carefully for all stages, to ensure that the efficiency of the pump remains high throughout the design duty period.

8.8.3 Pump Preservation

The economic life time of a water pumping equipment under normal operation with scheduled (at least every 3 months) planned preventive maintenance (PPM) is 10 but can also be stretched to 15 years. However, premature pump breakdowns and/or unsustainable loss of pump efficiency often occurs requiring pump replacements after 5 years of operation or less. This undesirable situation is normally due to cavitation, inadequate design, incorrect pump specification and installation, poor protection, operation and maintenance practices.

Some causes of failure that need to be considered during pump design and installation are as listed below:

i) Proper source selection, location and intake design is key in avoiding silting and chemical erosion. Avoid poor water quality sources or provide for erosion resistant intake structures in case of limited alternatives and disregard clear water pumps at such sources.
ii) Inadequate sump to pump connection results in turbulent flow and allows air entrainment in the pump causing cavitation. Also the use of complicated suction pipe lines that make priming difficult subjects the pump to bottle necks that lead to low net positive suction head (NPSH).
iii) Lack of sound hydraulic, mechanical and electrical protection can lead to motor burn-out due to overload, phase failure, under-voltage, lightning and switching surges. This leads to plant deterioration or damage and exposes operators to accidents.
iv) Poor installation can lead to pump damage and loss of efficiency. Problems due to poor installation include shaft cracks or breaks, burning of motors and vibration due to inadequate foundations. It is essential to ensure pump shaft alignment, adequate coupling clearance and proper foundation base plates fitting to avoid these issues.
v) Allow for intermittent electricity supply and provide for lightning arrestors to protect pumps against surges.
vi) Proper operation and maintenance should be observed as recommended below
    a. Technical level education required for plant operator and maintenance staff
    b. Adherence to planned preventive maintenance
    c. Adequate stock of likely spare parts required for emergency repairs
    d. Ensure sufficient attention to plant operation

Chapter 9

TREATED WATER STORAGE

9.1 General

The purpose of storing water in the distribution system is mainly twofold:

i) Balancing of variations in water demand during the day; and
ii) Emergency storage to ensure supply during break-downs or for fire-fighting.

The balancing storage requirement in a water supply system is caused by the cyclical variations in water demand over a period of time. In ordinary water supply systems, the usual period considered is one day. Figure**2-2** illustrates a typical daily water demand pattern in rural areas. If there is no facility for storing water for balancing purposes, the water source, raw water transmission main, treatment plant and treated water transmission main would all have to be designed to follow all fluctuations in water demand and this is generally not economical. Water storage reservoirs are provided first and foremost to balance the constant supply rate of the water source, raw water transmission mains, treatment plant, and treated water transmission main, with the fluctuating water demand in the distribution mains. The storage reservoirs have the overall effect of reducing the aggregate flows and thus the capacities, for which distribution mains have to be designed.

The emergency storage requirement is for meeting the water demand during periods of system breakdowns and fire fighting. The aggregate water volume, required for both balancing and emergency storage purposes can be kept in one or more storage reservoirs. The numbers and locations of the reservoirs to be provided should be determined after carrying out appropriate technical and economic analyses. The principal goal should be to minimize the total cost of the whole water supply system, including pumping stations, pipelines and storage reservoirs.

9.2 Balancing Storage

The required balancing storage volume for a distribution system is determined as follows:

i) The diurnal demand pattern for the supply area is estimated as illustrated in Figure 2-1 expressed in terms of percentages of the total demand over the whole day covered by the diurnal demand pattern and then plotted in a cumulative water demand curve (mass diagram) as shown in Figure 9-1. The constant water supply rate from the treatment plant and treated water transmission main is then drawn on the same diagram, as a straight line. The required balancing storage volumes can then be read off from the diagram as illustrated in Figure 9-1.
ii) In practice, for a 24-hour constant supply rate, the balancing storage volume provided will be of the order of 30% of the Maximum Day Demand of the area served. Hence, in the case shown in Figure 9-1, storage reservoir No. 1 will have a balancing storage capacity of 30% of the Maximum Day Demand of area E and storage reservoir No.2 will have a balancing storage capacity of 30% of the Maximum Day Demand of areas C and D. For constant supply rates of less than 24 hours, like 10-hour pumping, the required balancing storage volume can also be found from the cumulative water demand curve (mass diagram), as shown in Figure 9-1.

It is often more economical to analyse balancing storage volume requirements for different years, and then provide storage reservoirs in phases, because storage requirements will be quite low during the first few years of operation of a water supply scheme.

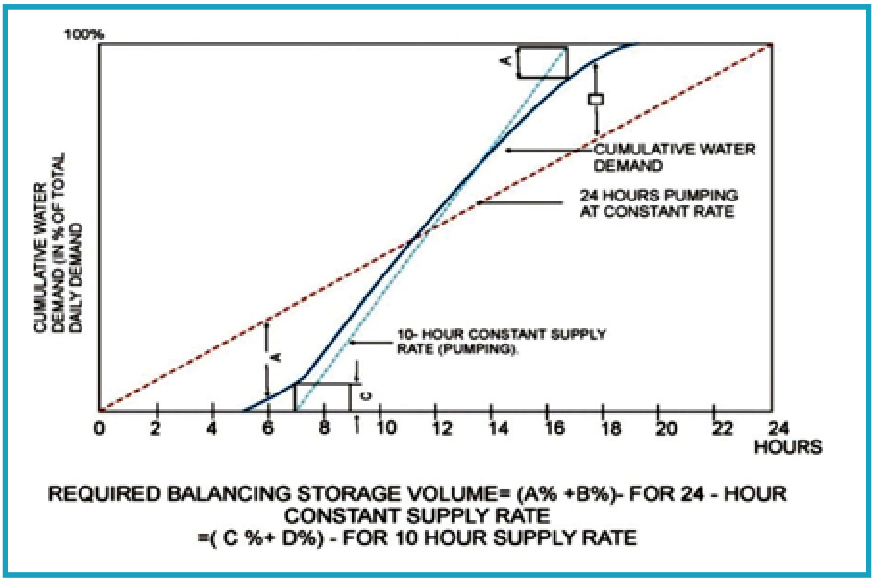


Figure 9-1: Graphical Determination of Required Storage Volumes.

9.3 Emergency Storage

In order to have enough water during the daily peak demand even in case of a major break upstream of the reservoir, it is usual to foresee an emergency storage volume corresponding to 2 hours of average hourly consumption. For smaller schemes, where no technical staff is available to carry out major repairs immediately, the emergency storageshould be higher.

In practice however, no special provisions are to be made for emergency storage in rural areas, except for institutions or industries which may provide their own emergency storage to safeguard against interruption of their supplies. Large towns and urban centers should have provisions for emergency storage volumes as shown in Table 9-1.

Table 9-1: Emergency Storage Volumes in Large towns and Urban Centres

Storage Reservoir Served by	No. of Hours of Maximum Day Demand
Gravity system	6
Pumped system	12
More than one independent system	0

The emergency storage volume should be put together with the balancing storage volume in one storage reservoir. The reservoir should be sited as close to the consumers it serves as is practicable. Large consumers who are dependent on a reliable water supply (like industries, hospitals and schools), should be encouraged to provide their own storage facilities.

9.4 torage Reservoir Design

9.4.1 General

The selection of storage reservoir materials largely depends on whether the reservoir is to be placed on the ground or to be elevated on a support structure. Ground storage reservoirs are normally made of reinforced concrete or concrete blocks or bricks. Elevated storage reservoirs are normally made out of galvanized pressed steel panels, and placed on steel support structures. All other factors being equal, the most economical storage reservoir shape is “circular”, followed by “square” and lastly “rectangular”. Normally, the depth of water in a storage reservoir should not exceed 5 m.

Reservoir sites should be selected such that they are on stable ground, not threatened by landslides or erosion. Level ground sites are preferable because they simplify excavation work. The design of storage reservoir foundations must be based on proper evaluations of ground bearing capacities.

9.4.2 Capacities

It is recommended that reservoirs are provided in standard capacities of 10, 25, 50, 100, 150, 200, 300, 500, 800 and 1,200 m^3.

9.4.3 Design Features

The size of the inlet line is determined by the water supply and demand requirements of the community and should have a shut off valve next to the reservoir. The size of the outlet is also determined by the demand requirements and should be located at least 200 mm above the reservoir bottom. This creates a dead volume which allows for settlement of sediment. It should also have a shut – off valve adjacent to the reservoir. The drain (washout) facilitates cleaning of the reservoir and is located at the bottom of the tank. Cleaning is done by shutting the main line valve and opening the discharge line. The reservoir is constructed with a slope towards the drain line to simplify cleaning. Reservoirs should have an overflow line that can support the maximum anticipated overflow (pump or spring capacity). The overflow pipes should be at least 50% larger than the inlet pipes and should be well screened and covered. It should be possible to observe the overflow so that the inlets can be controlled.

Manholes and covers are installed to provide an entrance during cleaning, maintenance and repair. They should be raised higher than the roof level to prevent entry of contaminated surface water. The manhole covers prevent entry of sun rays that encourage algae growth. They should be lockable.

Water level indicators are used to indicate the water level inside the reservoirs. A depth gauge using a float and wire is usually used. The type of control valves used will depend on the type of operation used by the system. The flow into the reservoir may be stopped manually or automatically by a float valve, pressure switch or any other device. Double orifice valves allow release of air when water is flowing while single orifice valves will allow release of air accumulated at specific points to be released when water starts flowing. Large reservoirs should be equipped with internal and external ladders and have external walkways and handrails, especially if they are elevated. They should be appropriately partitioned if the reservoirs exceed 500 m^3.

9.4.4 Structural Design of Reservoirs

The structural design of reservoirs will require expertise in the following categories:

i) Reinforced concrete storage reservoir design;
ii) Stonemasonry storage reservoir design;
iii) Steel plate storage reservoir design; and
iv) Plastic storage reservoir design.

Care must be exercised to ensure that the above aspects are done in accordance with the relevant guidelines and details are produced accordingly.

Chapter 10

ENVIRONMENTAL AND SOCIAL IMPACT ASSESSMENT (ESIA)

10.1 Introduction and Definitions

Environmental Impact Assessment (EIA) is defined, according to the Uganda National Environmental Act as "a systematic examination conducted to determine whether or not a project will have any (adverse) impact on the environment".

The EIA is generally used to accomplish the following goals:

i) To identify whether or not a proposed policy or project or activity is likely to have significant environmental impacts (both adverse and beneficial);
ii) If YES, to identify the potential significant environmental impacts;
iii) To analyze the significance of the adverse environmental impacts;
iv) To determine whether the adverse environmental impacts can be mitigated;
v) To recommend appropriate preventive and/or mitigation measures;
vi) To identify and assess any other alternatives to the proposed policy, plan, programme and project or activity; and
vii) To recommend whether or not the proposed policy, plan, programme and project or activity should be implemented or modified.

10.2 Policies, Legal, Regulatory and Institutional Framework

It is the firm policy of the Government of Uganda for ElA to be conducted for all planned policies, plans, programmes and projects or activities that are likely to have or will have significant impacts on the environment, so that the adverse impacts can be foreseen and eliminated or mitigated enhancing good impacts. It is also the policy of the Government for the EIA process to be inter-disciplinary and fully transparent, so that all stakeholders have access to the process, and that the process serves to provide a balance between the environmental, economic, social and cultural values, in order to ensure sustainable development in the country.

The supreme law in Uganda is the Constitution of the Republic of Uganda. The constitution requires the State to promote sustainable development and public awareness of the need to manage land, air and water resources in a balanced and sustainable manner for the present and future generations. In addition, Uganda is also a party to a number of international and regional agreements which requires her to comply with provisions of the agreements when developing policies, plans, programmes and projects. This is particularly mandatory when projects will be fully or partly funded by international funding institutions like the International Monetary Fund, World Bank, European Union, Asian development Bank, African Development Bank, East African Development bank etc.

10.2.1 Local Policy Framework

Table 10-1 presents policies and legal framework that should be complied with when conducting/ implementing an Environmental and Social Impact Assessment of water supply policies, plans, programmes and projects or activities.

Table 10-1: Policy Framework Relating to Water Supply Projects

Policy Title	Policy Goal	Relevance To Water Supply Projects
National Environment Management Policy, 1994.	To promote sustainable economic and social development that enhances environmental quality.	Environment Impact and Social Assessment (ESIA) must be conducted for water supply projects so that they promote economic and social development in a sustainable way.
The National Policy for the Conservation and Management of Wetland Resources 1995.	To curtail the rampant loss of wetland resources and ensure that benefits from wetlands are sustainable and equitably distributed. Wetlands acting as sources of water supply should be fully protected.	Application of environment impact mitigation procedures on all activities of the project to be carried out in or around affected wetlands. The project developers have to work hand in hand with Wetlands Inspection Division (WID) and NEMA to halt encroachment on wetland areas around the water sources where water is abstracted.
The National Water Policy, 1999.	Provides a framework for the protection of water, e.g. to discharge of effluents into surface waters, one must possess a permit before doing so.	Water supply projects may include construction of sewage treatment systems. The effluent from sewage treatment systems must be treated to meet effluent discharge standards so as not to pollute the receiving waters.
National Gender Policy, 1997.	Provides a framework and mandate for all stakeholders to address the gender imbalances within their respective sectors.	The gender policy recommends that integration of gender issues in national policies and projects will improve national welfare, contribute towards sustainable development, and improve the work of the ministries.

10.2.2 Local Legal and Regulatory Framework

Table 10-2 presents the relevant local legal and regulatory framework relating the water supply projects. Policies, programs, projects or activities.

Table 10-2: Legal and Regulatory Framework Relating to Water Supply Projects.

Act	Relevant Provisions
National Environment Act Cap 153	Section 19 (3), requires a developer of a project to submit an acceptable EIA Report in accordance with the guidelines in the Third Schedule of this Act.
The National Water and Sewerage Corporation Act, Cap 317	Section 3 of this statute states that the NWSC shall operate and provide water and sewerage in areas entrusted to it under the Water Statute of 1995.
Environmental Impact Assessment Regulations 13/1998 and Environmental Audit Guidelines	According to sections 19-20 of the NEA, all projects that have or are likely to have a significant impact on the environment are required to undergo an environmental impact assessment (EIA) process prior to implementation.

Act	Relevant Provisions
The National Environment (Wetlands, River Banks and Lake Shores Management) Regulations, 2000	Regulation 12(1) prohibits any person from carrying out an activity in a wetland without a permit issued by the Executive Director of NEMA.
The Water Act, Cap 152 and the Water Resources Regulations, 1998	- The Act provides for hydraulic works and use of water. -The Act provides for use, protection and management of water resources and supply; to provide for the constitution of water and sewerage authorities; and to facilitate the devolution of water supply and sewerage undertakings.
Water (Waste Discharge) Regulations, 1998	The water (Waste Discharge) Regulations of 1998, are aimed at regulating the effluent or discharge of wastes on to land or into water.
The Land Act, Cap 227	Section 42 states that Government or Local Government may acquire land in accordance with the provisions of Article 26 and clause 237 of the constitution.
Occupational Safety and Health Act, 2006	The Act aims at ensuring the existence of safety and health at all work places and work environment.
Workers' Compensation Act Cap 225	This requires compensation to be paid to a worker who has been injured or acquired an occupational disease or has been harmed in any way in the course of his/her work.
National Environment (Conduct and Certificate of Environment Practitioners Regulations (2003)	Regulation 176 (1) states that no person shall conduct and EIA or carry out any activity relating to the conduct of an environmental impact study, or environmental audit as provided under the Act, unless the person has been duly certified and registered in accordance with the regulations
The National Environment (Waste Management) Regulations, 1999	Regulation 9 (8) stipulates that at any reasonable time, an environmental inspector can: (a) stop and inspect any vehicle used for transportation of waste; and (b) enter upon any premises where waste is stored.
The National Environment (Control of Smoking in Public Places) Regulations, 2004.	Section 3 entitles every person to a healthy environment, free from second-hand smoke. It further obliges all persons to safeguard the health of non-smokers. Sections 4 & 5 prohibit smoking in public places. During implementation of water supply projects and in their operation and maintenance, smoking workers should not inconvenience non-smoking workers. It thus implies that no smoking signs should be placed in various areas and it should be enforced.
The National Environment (Noise Standards and Control) Regulations, 2000.	Regulations 6 & 7 (II) sets permissible noise levels, Part III (Regulations 8, 9, 10 & 11) calls for the control and mitigation of noise; Regulation 9 specifically prohibits the generation of noise by place and time. Part IV instructs for a license for noise in excess of permissible levels.
The Town and Country Planning Act Cap 246	The Town and Country Planning Act 1964 govern land use and land planning in urban and rural areas. Thus, land acquisition for water supply projects should be done in accordance with this act.

Act	Relevant Provisions
Public Health Act Cap 281	Section 7 provides local authorities with administrative powers to take all lawful, necessary and reasonable practicable measures for preventing the occurrence of, or for dealing with any outbreak or prevalence of, any infectious, communicable or preventable disease, to safeguard and promote the public health.
The Local Governments Act Cap 243	Provides for the system of local governments based on the decentralization of district for the enforcement of environmental law. The functions of the Municipal Councils include: land surveying and administration, physical planning, environmental protection (forests and wetlands, streams etc.) and ensuring proper sanitation.

In addition to the above local legal framework, it is important to note that various local governments and/or urban authorities may have their own byelaws that apply in their areas of jurisdiction. Consequently, EIA practitioners should capture such byelaws relevant to the development of water supply projects.

Table 10-3: Example of World bank/IFC Legal Requirements for Water Supply Projects

Policy Title	Key Issues
Environmental Assessment Policy (OP/BP 4.01)	Preventing, minimizing, mitigating or compensation for adverse impacts caused by the project.
Natural Habitat Policy (OP/BP 4.04)	Promotion of environmentally sustainable development.
Pest Management (OP 4.09)	Prevention of pollution to water sources when using pests in agriculture
Involuntary Resettlement Policy (OP 4.12)	Avoid involuntary resettlement and assist displaced persons in restoring their livelihoods and living standards.
Access to Information Policy (2010)	Promote raising awareness about development issues, share global knowledge, and ensure participation in Bank programs and projects

Table 10-4: Some International Treaties and Conventions Relevant to Water Supply Projects.

Treaty Title	Key Issues
Kyoto Protocol to the United Nations Framework Convention	Promotion of sustainable forest management practices, afforestation and reforestation, especially on river banks and lakeshores which often serve as surface water sources.
United Nations Convention to Combat Desertification in Those Countries Experiencing Serious. Drought and/or Desertification, Particularly in Africa (UCCD, 1992)	Integration and sustainability of natural resources, promotion of alternative sources of energy and alleviation of pressure on fragile natural resources.

Treaty Title	Key Issues
Stockholm Declaration (Declaration of the United Nations Conference on the Human Environment 1972)	Principle 15 of the Stockholm Declaration states that, "Planning must be applied to human settlements and urbanization with a view to avoiding adverse effects on the environment and obtaining maximum social, economic and environmental benefits for all". In this respect, projects aimed at exploiting local people or the environment is discouraged.
EAC Treaty	Promotion of clean and healthy environment is a prerequisite for sustainable development.

The above tables give examples of local policies and regulatory framework as well as examples of legal requirements, international treaties and conventions and are not necessarily exhaustive and exclusive. Consequently, the EIA practitioners should look out for specific local laws, international treaties and conventions that apply to specific water supply projects. It must be noted that both local and international legal and regulatory requirements are not static and evolve from time to time. Therefore, EIA practitioners should always seek out to find and make use of the most applicable and up to-date legal and regulatory documents.

10.2.3 Institutional Framework

The NEMA, Cap. 153 Section 4 established the National Environment Management Authority (NEMA) to be the principal agency in Uganda for the management of the environment and to coordinate, monitor and supervise all activities in the field of the environment. According to NEMA, Cap. 153, Sections 19-20, the project developer must submit to NEMA a project brief that may form the basis of approval of the project, directly, or through an environmental impact review and if insufficient, an environmental impact evaluation will have to be done. If the NEMA deems the project brief to be complete, with sufficient mitigation measures for any anticipated impacts, then NEMA will transmit the project brief to the lead agency for comments, where the lead agency means a Ministry, department, parastatal agency, local government system or public officer in which or in whom any law vests functions of control or management of any segment of the environment. NEMA shall consider the project brief and comments from the Lead Agency and take a decision of approval (with/or without conditions) or disproval and inform the developer of the next steps.

For projects needing a full-fledged Environmental and Social Impact Assessment (ESIA), the entire process starting from the submission of the project brief, through screening, EIStudy (Scoping, production and approval of ToRs and persons to conduct the ESIA), submission and review of the Environmental and Social Impact Statement is done in close consultation with NEMA, lead agency and stakeholders according to the schematic presented in Figure 10-1. The persons carrying out an EIA/ESIA must be duly paid up members of the Uganda Association for Impact Assessment (UAIA) and must be duly registered and certified by NEMA.

All environmental and social impact assessment must have an Environmental Monitoring Plan (EMP). It must be noted, that the approval of ESIA may be done with or without conditions, and the developer must implement and meet the conditions of approval of the ESIA. If the ESIA is approved without conditions, it means that the EMP has been accepted as submitted in the EIA/ESIA. The developer must comply with the activities outlined in the EMP and the conditions of offer. NEMA may cancel the EIA/ESIA if the developer fails to comply with the EMP and conditions of offer as outlined in the approval certificate or if there has been substantial modification of the project implementation or operation, which leads to adverse environmental impacts; or if there is substantive undesirable effect not contemplated in the approval.

One of the conditions of approval of EIA/ESIA is the requirement by the developer to carry out self-environmental audit, normally after every year. NEMA is also mandated by the NEMA to carry out environmental audit of projects during the implementation and/or operation phases. In carrying out auditing, if the developer fails to implement mitigation measures suggested in the EMP as well as those that may be enlisted in the conditions of approval of EIA/ESIA, an environmental inspector working on the behalf of NEMA may issue against the developer, an improvement notice and commence such criminal and civil proceedings provided for under the NEMA.

10.3 Levels of EIA/ESIA for Projects

The EIA/ESIA process required for a proposed project shall be appropriate to the nature, scale and possible effects of the project and also to the nature of the proposed project site. The levels of EIA/ESIA required for proposed projects will vary on a project-to-project basis, but in general such levels will fall into any of the following three major categories:

i) Small-scale projects having potential adverse environmental impacts which can easily be identified, and for which mitigation measures can readily be prescribed and included in the design and/or implementation of the project. The environmental aspects of such a small-scale project would normally be approved on the basis of the mitigation measures prescribed in a project brief, without the need for a detailed Environmental Impact Study (EIStudy). Such projects may include: emergency repairs to a water supply scheme within the character of its surroundings, water supplies to individual subsistence small farms, extensions of water supply schemes to localised communities within a water supply area, development of localised water points such as protected springs, shallow/hand dug wells, boreholes for small rural communities, onsite sanitation facilities and treatment plants for small communities *e.g.* public toilets, septic tanks, bio-latrines, baffled reactors.

ii) Projects for which there is some level of uncertainty regarding the nature and level of environmental impacts expected, thus requiring a more in-depth Environmental Impact Review (EIR) to determine if appropriate mitigation measures can be identified and prescribed, or if a more detailed EIStudy would be required. If during the EIR it is found that adequate mitigation measures can be identified and incorporated in the project design, then the necessity for detailed EIStudy may be eliminated and the environmental aspects of the project may be approved. The EIR carried out at this stage would normally examine various alternatives, so that the decision maker can select options which do not have significant environmental impacts. Projects that fall in this category include small scale gravity water supply schemes, localised small scale valley dams/tanks, water supply and wastewater treatment plants for medium sized communities.

iii) Projects which clearly will have significant environmental impacts for which mitigation measures cannot readily be prescribed unless detailed EIStudy of the projects and their possible alternatives is conducted, with a view to determine if there are other alternatives which have less adverse environmental impacts. Conducting such EIStudy requires greater public participation. Projects that fall within this category include water supply systems for rural growth/urban centres and their components *e.g.* water intake structures abstracting water from wetlands, rivers or lakes; water treatment plants, water transmission and distribution systems, water pumping/booster stations and reservoirs/water storage tanks, wastewater collection pipes, waste stabilisation ponds/lagoons and conventional wastewater treatment plants.

10.4 Methodology for Conducting EIA/ESIA

10.4.1 General

The basic components of the EIA/ESIA process in Uganda consist of the following three inter-connected phases:

i) Phase I: Screening;
ii) Phase II: Environmental Impact Study (EIStudy); and
iii) Phase III: Decision making.

Figure 10-1 presents the interconnected phases that have to be followed in the EIA process, according to the National Environment (Environmental Impact Assessment) Regulations, Statutory Instrument 153-1.

10.4.2 Phase I: Screening

The objective of the screening phase is to determine if a proposed project does or does not have significant environmental impacts. If a project is found not to have potential to cause significant environmental impacts, it shall be categorically excluded from further environmental impact assessment, and a decision shall be made to approve and implement the project, with recommendations to the project developer for sound environmental management of the project.

If, however, the project is found to have the potential for significant environmental impacts, further screening should be conducted to determine if adequate mitigation measures can readily be identified and prescribed through further EIR, or if full EIStudy will be required. If in conducting the EIR it is found that adequate mitigation measures can be incorporated in the project design, then the environmental aspects of the project can be approved. If, on the other hand, adequate mitigation measures cannot be readily identified, then the project shall be subjected to further detailed EIStudy (Phase II).

If a decision is made at the screening stage to exempt a project from detailed EIStudy, or to approve its environmental aspects on the basis of identified and prescribed mitigation measures, such a decision shall be contained in a "Certificate of Approval of the Environmental Impact Assessment" issued by the National Environmental Management Authority (NEMA). If, however, after screening it is determined that a project requires detailed EIStudy, then the certificate of approval shall only be issued after approval of the Environmental Impact Statement (EIS) emanating from the detailed EIStudy conducted.

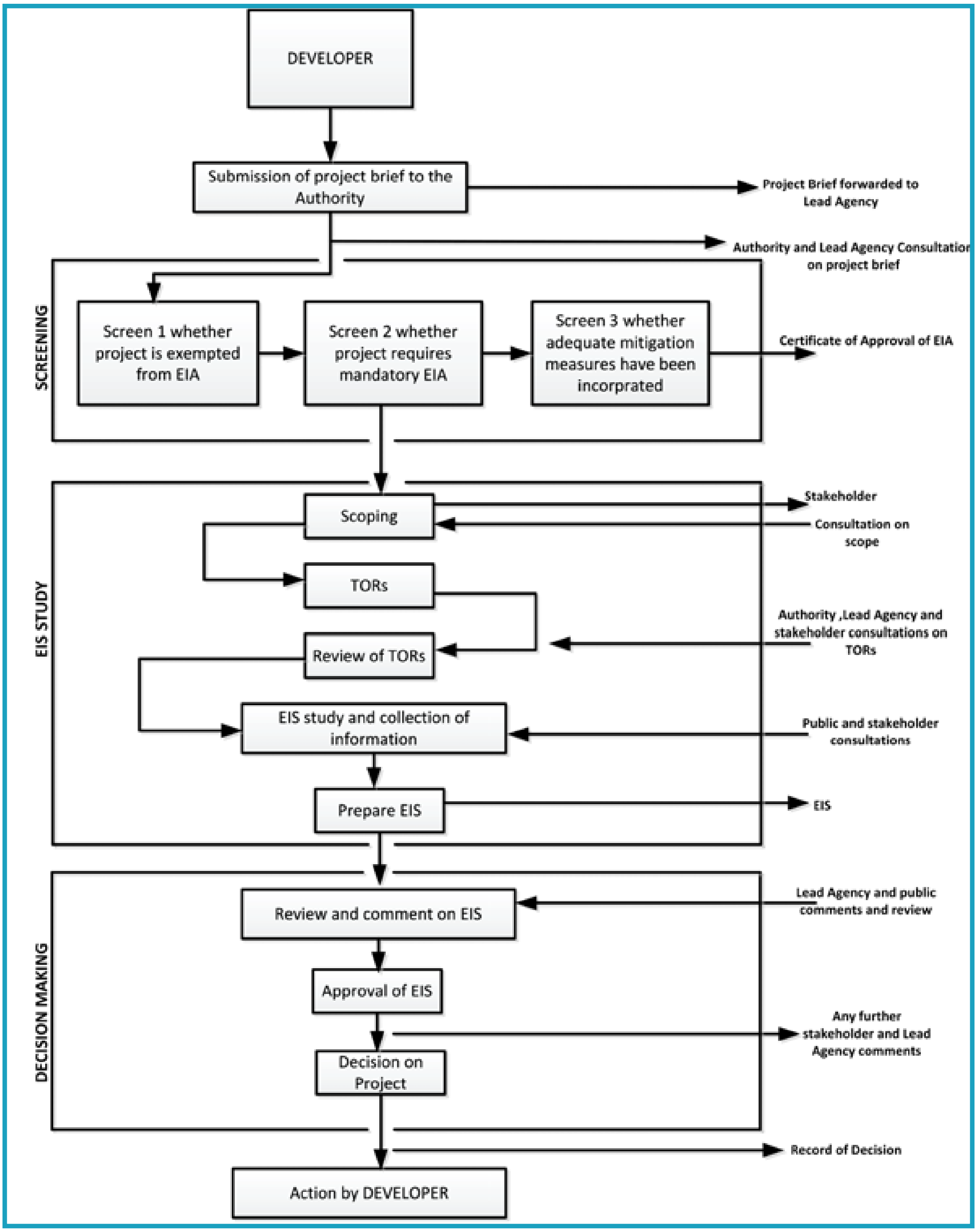


Figure 10-1: Schematic Representation of the EIA Process (Source: EIA Regulations, 2005).

10.4.3 Phase II: Environmental Impact (EIStudy)

10.4.3.1 Scoping

The first step in the EIStudy of a proposed project is to determine the scope of work to be undertaken in assessing the likely environmental impacts of the project. "Scoping" involves the identification of potentially significant environmental impacts, and it is to be applied to all activities which require full EIStudy. Usually, this exercise includes meetings and consultations with all the project stakeholders, to obtain their views on what should be included in the EIStudy, and what alternatives should be considered in order that an adequate EIStudy is conducted.

The responsibility for "scoping" shall be that of the project developer, but the NEMA and other interested parties shall be consulted about the exercise. The purpose of the scoping exercise is to inform the development of the ToR for the EIStudy.

The project developer shall prepare a "scoping" report summarizing the results of the scoping exercise. The scoping report shall, together with the Terms of Reference (TOR) for the EIStudy be submitted to NEMA. The NEMA shall in turn study the ToR and also forward the TOR to the appropriate concerned agencies for their comments. NEMA shall then approve the ToR with or without amendments and communicate to the developer of the decision.

The "scoping report" and the TOR shall cover the following main items:

i) Delineation of the boundaries of the area to be covered in the EIStudy.
ii) Questions about the proposed project which should be answered through the EIStudy.
iii) Identification of the potentially significant environmental impacts of the project, which should be addressed in the EIStudy.
iv) Alternatives to the proposed project.
v) The full range of the project stakeholders and relevant agencies affected by and interested in the project, which are to be consulted and involved in the EIStudy process.
vi) Other relevant technical aspects related to the proposed project.
vii) Identification of other projects in the area that may be impacted upon by, or will impact on the proposed project.
viii) How the proposed project conforms to the existing laws, policies and regulations.

The identification of potentially significant environmental impacts is left to the discretion of all the parties involved in the "scoping" exercise. In identifying potentially significant environmental impacts, participants in the "scoping" exercise should use their own experiences, expertise and knowledge of the project and its area/site. Alternatively, suitable standard checklists of potential significant environmental impacts can be used.

10.4.3.2 Scoping Checklist

For water supply projects, the following checklist which is not necessarily exhaustive can be used:

i) Pollution

a) Air Pollution from construction and water treatment chemicals

b) The construction of water supply unit processes as well as wastewater treatment unit processes, reservoirs, water/wastewater conveyance systems and other appurtenances may produce airborne dust that may compromise the quality of the air. Wetting work surfaces may be done to minimize air pollution. Also, the storage and injection facilities of chlorine and other water treatment chemicals should be designed and operated in such a way as not to endanger the operation staff and the environment in general.

c) Soil Erosion from Construction Works

d) The erosion, drainage and deposition of soil can occur as a result of clearance of top soil and removal of vegetation during the construction of a water supply scheme. These processes may result in the deterioration of the quality of the natural water resources downstream of the construction sites.

e) Noise and Vibrations

f) Usually, there are excessive noise and vibrations around water treatment plants and pumping stations. Appropriate buffer zones or sufficient distances between the sources of noise and vibrations and the habited areas should be provided.

g) Deterioration of Water Quality in Dam Reservoirs

h) In water supply projects where dam reservoirs are used, water quality deterioration in the reservoirs including detrimental changes in water temperatures caused by household and industrial effluents, and the consequent impacts on the water supply, fishing and other water uses, should be studied. Appropriate mitigation measures should make reference and use of the concept of water safety plans, where end product testing is not the emphasis, but rather, the identification of critical control points that cause pollution to the water sources should be undertaken with the aim of preventing the pollution from happening.

i) Change in Water Quantity from abstraction points *e.g*. wetlands, swamps, rivers, ground water

j) Climate change effects that may result into seasonal changes in water quantity of water sources should be evaluated. For surface water sources, *e.g.* rivers and streams, the abstraction should be done such that even in periods of extreme low flows *e.g.* during draughts, there is an acceptable minimum environmental flow that should be maintained in the river/stream. The abstraction of water of water from any water source should not compromise other legible water users along the river/stream i.e. downstream of the abstraction point. Also, indirect and cumulative impacts should be evaluated, e.g., the sinking of boreholes should not lower the water table to result into result into die-off of vegetation miles away. Local knowledge on the historical low flow levels and maximum flood levels may be useful in the evaluation.

k) Sludge from Water and Wastewater Treatment Plants

l) Sludge produced in water treatment plants, should be treated and re-used if it meets the criterion for re-use or disposed of in an appropriate manner, such as landfilling if it is not fit for any re-use. The potential re-use options for sludge may be anaerobic digestion to produce biogas for cooking and lighting as well as slurry that can be used in agriculture; composting for the production of soil conditioner and fertilizer; or other valorization techniques that may include the production of animal fodder, energy fuel, pyrolysis for the production of bio-charcoal etc. The treatment and final re-use of the sludge, or ultimate disposal, should be designed and implemented in a way that minimizes greenhouse gas emissions to the atmosphere.

ii) Natural Environment Issues

a) Effects of the Project on the Ecology

The effects of the project facilities such as intake structures, treatment works, pipelines and access roads, on the natural flora and fauna habitats in the project area, should be studied. Special emphasis should be put on any fauna and flora species that have been locally or internationally declared to be endangered. Projects to be developed should not negatively impact on endangered species. Any other flora or fauna specifies, which are not endangered should be relocated. This may also involve the re-planting of lost flora species with similar species after the activity that resulted into their loss has been completed, *e.g.* filling up and replanting borrow pits and quarry sites.

In schemes using dam reservoirs, the effects of water quality deterioration in the reservoir and reduced volumes of water flowing downstream, on the fish, animals and vegetation in the watershed area should also be studied.

b) Effects on the Landscape

The impact of the project on the unique scenic beauty of the local landscape and on the main viewpoints in the area should be studied.

c) Ground Subsidence

In water supply projects utilizing groundwater sources, ground subsidence caused by excessive pumping of groundwater must be avoided.

iii) Human Environment Issues

a) Effects on the Historic and Cultural Heritage Sites

The project facilities should not be located in areas where they interfere with or cause damage to the important historic and cultural heritage sites in the area.

b) Effects on the Existing Infrastructure

The effects of the project on the existing service infrastructure such as buildings, roads, railways, bridges and river traffic should be studied.

c) Relocation of Populations

In the case of dam reservoir construction, the situation and circumstances regarding land inundation and the local residents who have to be relocated as a result, should be assessed.

d) Occurrence of Water-related Diseases

Particular attention should be paid to the risks of occurrence of water-related diseases (such as malaria and schistosomiasis) as a result of the introduction in the area of water supply facilities such as dam reservoirs.

e) Effects of Construction Work

In projects involving major construction work, the effects of noise, dust, offensive odours and vibrations from the work should be studied.

10.4.4 Public Involvement During Scoping

To ensure quality, comprehensiveness and effectiveness and in order to adequately address stakeholder views, there shall be public involvement during scoping and this will be involve the following:

i) Notification/invitation for public comment and written submissions;
ii) Consultation with the various stakeholders;
iii) Workshops and facilitated discussion
iv) Public and community meetings; and
v) Use of communication media.

10.4.5 Conducting and EIStudy

i) General

The information gathered during the "scoping exercise" and contained in the "scoping" report and the Terms of Reference for the EIStudy, shall form the basis for conducting the study. The terms of reference shall be approved by NEMA before the EIA/ESIA is conducted. An approval letter of ToR shall be issued by NEMA.

The study team shall comprise of persons who are registered and certified by NEMA to carry out environmental impact assessments. The study team will also be members of the Uganda Association for Impact Assessment (EIA). The study team shall endeavour to collect all the relevant data needed to address the significant environmental issues identified during the "scoping exercise". During the study, the team shall conduct field work and consult widely with all the project stakeholders, relevant agencies, and the general public to obtain and verify information such as:

a) Proof of land ownership for the development – by means of sale agreements, certificate of land title or other legally binding documents,
b) Baseline information that will include: socio-economic status of the project affected persons (PAPs) and their immediate surroundings, topography, climate, water resources, hydrogeology, geology and soils, flora and fauna.
c) Information on cultural heritage and other areas of historical significance,
d) GPS locations of the land and key points, e.g. the sampling points for soil and water quality analysis,
e) A Google map or other appropriate map of the land where the development is to be carried out, so that it is easier to access such a site by environmental inspectors.

The EIA/ESIA study team will also inquire about the preliminary impacts that can be collected by physical field observation and stakeholder consultations, and then study and analyse the information collected to compile all potential impacts of the project. Mitigation measures for adverse environmental impacts shall be proposed, and compensatory measures recommended for unavoidable impacts. It is essential that the cost-effectiveness of the proposed mitigation measures is analysed against other viable alternatives.

When presenting impacts, they should be categorised based on the severity of the potential impact using a predefined impact rating criteria as minor, moderate or major according to the definition below.

a) Minor impact – an effect will be experienced, but the impact magnitude is sufficiently small and well within accepted standards, and/or the receptor is of low sensitivity/value.
b) Moderate impact – an impact that will be within accepted limits and standards. They may vary from a threshold below which the impact is minor up to a level that might be just short of breaching an established regulatory limit.
c) Major impact – is where an accepted limit or standard may be exceeded, or large magnitude impacts occur to highly valued/sensitive resource/receptors.

Extent: within limited area (<200 m from site), local (up to 10 km) or wide (regional or global > 10 km); Duration: temporary (1 year), short term (1-5 years), medium term (5 -10 years) long term (> 10– 50 years) or permanent.

A tabular presentation is preferred, *e.g.* the following impacts (Table 10-5) were identified in the ESIA for Arua Water Supply Expansion and Sanitation Project. The potential impacts presented in Table 10-5 are for just one component (water treatment works) and one phase (construction) of the project. Therefore, impacts have to be identified for all project components and phases, including operation, maintenance and decommissioning phase.

Table 10-5: Potential Impacts for the Rehabilitation of Water Works.

Project component: Water Treatment works			*Phase: Construction phase*		
	Issue	**Potential Impact**	**Type and Rating**	**Extent**	**Duration**
1	Air pollution	Emissions from construction equipment and project vehicles.	Direct, Minor	Limited	Temporary
2	Noise pollution	Intermittent noise from construction. Equipment and heavy project vehicles.	Direct, Moderate	Limited	Temporary
3	Water pollution	Water pollution from dredging activities, accidental spillage of fuel and lubricants.	Direct, Major	Limited	Temporary
4	Water levels	Water levels may be affected by rehabilitating the impoundment weir and reservoir.	Direct, Major	Wide	Short to Long term
5	Soil erosion and contamination	Inappropriate construction practices and soil protection measures which may induce or accelerate soil erosion with possible pollution and siltation of downstream water sources; Removal of top soil may lead to loss of soil fertility.	Direct, Major	Limited	Temporary
6	Solid waste generation	Domestic waste from camps be an eye sore and may contaminate soil and water resources.	Direct, Moderate	Limited	Temporary
7	Impacts on flora and fauna	Loss of wetland plants and associated fauna; Cleared vegetation may compromise aesthetic value of the sites.	Direct, Minor	Limited	Temporary
8	Public Safety including accidents as a result of increased vehicular traffic	Excavations and transportation of equipment, site workers and debris and movement of heavy equipment may pose a safety risk to the general public; Increase in the likelihood of accidents within and around the vicinity of water works area from possible careless driving of project vehicles.	Direct, Moderate	Limited	Temporary
9	Public health problems	Pools of stagnant water may form in pits, holes and excavated ditches and create suitable habitats for disease vectors such as malaria; Potential of HIV spread as well as poor hygiene in workers camps.	Direct, Moderate	Limited	Temporary

Project component: Water Treatment works			*Phase: Construction phase*		
	Issue	**Potential Impact**	**Type and Rating**	**Extent**	**Duration**
10	Occupational health and safety	Exposure of workers to occupational health and safety hazards from activities such as: excavations; working with heavy equipment; working under noisy conditions, working in confined spaces; lifting of heavy objects; storage, handling and use of hazardous substances and wastes; Poor hygiene and sanitation in workers camps.	Direct, Moderate	Limited	Temporary
11	Disturbance/ interruption of commercial and social activities	Interference with commercial and social activities.	Direct, Minor	Limited	Temporary
12	Disruption of social order	Influx of people in the area which may affect the local economy, cause alteration of culture and introduce behavioral changes.	Direct, Minor	Limited	Temporary
13	Raw material use	Large quantities of construction material will be involved, for example, cement, steel, oil fuel, pipe materials (e.g. PVC, uPVC, concrete and/or steel). Also, large quantities of local materials, *e.g.* sand, gravel will be involved. Additional impacts include wet season excavation, creation of quarry sites and borrow pits. If excavated areas are not re-instated; and if materials are not well stored and utilized, as well as instituting management measures for waste materials, contamination of the environment may occur.	Direct, Moderate	Limited	Temporary
14	Visual amenities	Construction sites, if not well managed, may have impacts on aesthetics of the surroundings with the possibility to affect the neighboring residents to the WTW with moderate view point.	Direct, Minor	Limited	Temporary

Source: *ESIA for Arua Water Supply Expansion and Sanitation Project, 2012.*

ii) Analysis of alternatives

The EIA process seeks to compare various alternative options which may be available for a project, and thus determine which alternative represents the most desirable balance between environmental and economic costs and benefits. The EIA process shall, therefore, include an analysis and discussion of a reasonable range of alternatives to the proposed project, which could feasibly meet the basic objectives of the project.

In the analysis of the alternatives, the environmental losses and gains associated with each alternative should be compared together with the economic costs and benefits, to provide a full and balanced picture of each alternative. A recommendation of the preferred alternative and the reasons why it has been chosen shall be given in the discussion of the alternatives.

iii) Impact Mitigation

One of the main objectives of the EIA process is to predict and prevent unacceptable adverse environmental effects of a proposed project, by recommending appropriate project modifications or actions which reduce, avoid or offset the potential adverse environmental consequences of the project. To achieve this objective, it is necessary for the EIA to be carried out at the same time as the project design, so that any design changes can be made early enough.

This approach has the advantage of saving the project developer time and money which would otherwise be wasted if the changes were to be made later on after project design has been finalized.

The purpose of mitigation is to identify alternative and better ways of implementing the proposed project so that the negative environmental impacts are eliminated or minimized, while the benefits are enhanced. Impact mitigation, however, is only possible if the full extent of the anticipated environmental problems is understood.

Successful impact mitigation demands that the necessary mitigation measures are implemented at the correct time and in the correct manner. This usually requires a clear and agreed-upon monitoring plan, to ensure the actual implementation of the mitigation proposals.

iv) Monitoring

Procedures for monitoring the environmental performance of a proposed project shall be incorporated in the EIStudy. The tool used for monitoring post- construction, construction as well as operation and maintenance of the water supply project is the Environmental Management Plan (EMP). The EMP should present each impact type, the actual impact and mitigation measures, as well as a monitoring indicator, estimated unit cost, estimated total cost, the responsible entity/agency for mitigating the impact as well as the monitoring institution. The EMP should be presented for the pre-construction, construction, operation and maintenance and decommissioning phases. Project developers, whose projects have been subjected to EIStudy, shall ensure that the mitigation measures and actions to protect the environment are adopted and actually implemented. The developer shall undertake to conduct “self-monitoring”, “self-record-keeping” and “self-reporting”, and the information gathered through this monitoring process, shall be stored and made available during the inspection of the project by the NEMA.

10.5 Social Impact Assessment

10.5.1 General Policy

The section will be written by use of mainly secondary data and supplemented with interviews from selected water supply systems. The purpose of the interview will be to find out their experiences with social issues in the design of the systems. How they planned or not planned for them, the consequences and lessons learnt for future planning.

10.5.2 Introduction

Planners and decision makers increasingly recognize the need for better understanding of the social consequences of policies, plans, programmes and projects (PPPPs). Social Impact Assessment helps in understanding such impacts. Social Impact Assessment alerts the planners as to the likely benefits and costs of a proposed project, which may be social and/or economic (Vanclay, 2002). The knowledge of these likely impacts in advance can help decision-makers in deciding whether the project should proceed, or proceed with some changes, or dropped completely. The most useful outcome of a SIA is to develop mitigation plans to overcome the potential negative impacts on individuals and communities. SIAs can assist advocacy groups as well. A Social Impact Assessment report, done painstakingly, showing the real consequences of the project on affected people and suggesting alternative approaches, gives credibility to their campaigns (Council for Social Development New Delhi, 2010).

Water projects will normally comprise of the construction of new water treatment works, water reservoirs and extension of distribution mains in which the impacts are site specific. Few, if any, of the impacts are irreversible; and in most cases, the mitigation measures can readily be designed. Displacement of persons and assets is most unlikely.

10.5.3 Justification for Doing a Social Impact Assessment

While there is not a single precise definition for SIA, it has been defined as “the process of assessing or estimating in advance the social consequences that are likely to follow from specific policy actions or project developments” (Barrow, 2000; Joyce and Macfarlane, 2001). As well as social impacts, SIA may also assess the cultural, demographic and economic consequences of a proposal on all major stakeholders (Burdge, 2004). Social impacts include all social and cultural consequences to human populations of any public or private actions that alter the ways in which people live, work, play, relate to one another, organize to meet their needs, and generally cope as members of society. Cultural impacts involve changes to the norms, values, and beliefs of individuals that guide and rationalize their cognition of themselves and their society.

In essence, an SIA is conducted to address the question of “*Who benefits and who loses?*” (determining impact equity) from the implementation of a proposal (Barrow, 2000; Wolf, 1983). Burdge states that a properly done SIA should also answer the following questions:

i) What will happen if a proposed action was to be implemented – why, when and where?
ii) Who is being affected?
iii) What will change under different alternatives?
iv) How can adverse impacts be avoided or mitigated and benefits enhanced

10.5.4 Steps in Doing a Social Impact Assessment

10.5.4.1 Scoping

Scoping is the first step in social Impact assessment. Essentially, this will involves a visit to the project site, and consultation with all stakeholders. It is important to confirm their understanding of key issues. This allows for on-site appreciation of impacts especially for projects that cause displacement on a large scale. The local knowledge can be invaluable in finding alternatives that help avoid or at least reduce the magnitude and severity of adverse impacts. It is a stage to identify potentially impacted people; identify limits; decide on methodology, variables and data sources (Barrow, 2000, Finsterbusch et al., 1983; Wolf, 1983). Scoping should only identify all issues and affected groups to get all the cards on the table.

Factors that may be identified in the scoping will include:

i) Lifestyle, amenity and recreational use and access;
ii) Sense of place;
iii) Indigenous communities;
iv) Existing and future needs;
v) Equitable access to water; and
vi) Development footprint.

10.5.4.2 Profiling

This stage will determine who is likely to be impacted (stakeholders), establish current social profile and baseline data. To assess the extent of social impacts, it is necessary to assess the socio-economic conditions of the affected people. This assessment generally involves conducting a socioeconomic survey and a broad based consultation with all affected groups. The socioeconomic profiling should not be restricted to adversely affected population. The survey should include those who benefit from the employment and other economic opportunities generated by the project.

10.5.4.3 Prediction

The projection of impacts will use the information gathered from previous steps to predict what will happen and who will be affected. Direct social impacts are defined as those that result “directly from the social change processes that are invoked by a project” and may be either intended or unintended. It is important to remember that the impacts identified should be those which are expected to occur as a result of the proposal being implemented and not as a result of baseline trends.

Some of the possible positive impacts of water supply projects

i) Per capita water supply will increase;
ii) Timely and sufficient supply of quality water;
iii) Water borne disease will decrease and health status will improve;
iv) Employment opportunities to unskilled and semi-skilled persons during construction and service to a few persons in O&M;
v) Equitable and convenient distribution of water;
vi) Effective monitoring;
vii) With extensive and adequate supply pressure on ground water will decrease;
viii) Water recycled will be used for industrial and commercial purposes;
ix) Water sale to industries will add to revenue of water supply institutions; and
x) Employment opportunities available to tradesmen and suppliers of relevant services.

Possible Negative Effects

i) Land acquisition and displacement;
ii) Influx of people in the area;
iii) Public disruption due to traffic congestion etc.; and
iv) Diseases like HIV/AIDS.

10.5.4.4 Evaluation

This step will determine the magnitude and effect of impacts; determining the potential for avoidance/ mitigation; determine significance of identified impacts; and determine who benefits and who loses; evaluate whether overall impact is acceptable and select an option. Since many impacts are not quantifiable, it is impossible to rank them objectively. The community perceptions of an impact and those of the SIA team are not necessarily the same. The affected people should therefore be consulted in ranking impacts. If impacts are found unacceptable, the SIA must clearly state that giving reasons. Generally, the Social Impact Assessment is expected to result in specific mitigation plans to address relevant social/resettlement issues and potential impacts.

10.5.4.5 Mitigation

At this stage, measures will be identified to counter unwanted impacts. This step in SIA is to develop a mitigation plan to firstly avoid displacement, secondly to minimize it, and thirdly to compensate for adverse impacts. The major contribution of an SIA study is to help plan to, manage, and then mitigate any negative impacts (or enhance any positive ones) that may arise due to a proposed project. Possible mitigation measures include:

i) Land acquisition minimum to meet the project requirements,
ii) Land acquisition?
   - o Eligibility criteria and entitlement framework;
   - o Estimate affected families;
   - o Valuation of affected assets;
   - o Organizational procedures;
   - o Implementation framework;
   - o Redress of grievances;
   - o Mechanism of consultations; and
   - o Institutional arrangements for monitoring and evaluation.

iii) Safety and security to public during construction;
iv) Road and land will be restored to original condition after construction works;
v) Replacement by new plantations against the cutting of trees, wherever unavoidable;
vi) Access to emergency services- ambulance, police etc. to people during construction period;
vii) Commitment to engage in extensive consultation with landowners along the potential pipeline corridor prior to finalizing the route;
viii) Commitment to work jointly with communities to manage the potential impacts of influx of construction workers during the construction phase;
ix) Commitment to monitoring and transparent reporting of monitoring results;
x) Commitment to monitor social commitments;
xi) Commitment to open dialogue with local people and to offer to work jointly on agreed priority projects of benefit to both; and
xii) Commitment to develop a sustainability initiative.

10.5.4.6 Monitoring

This will identify during Project implementation what may happen, who is affected. Identify cause effect linkages and feedbacks, Predict indirect and cumulative impacts. This has to be done in two ways as follows:

i) Internal Monitoring

The responsibility for tracking progress towards achievement of project output and outcome will lie with the unit responsible for implementing a certain project or program – in many cases the executing or implementing agency. The responsible unit needs to allocate sufficient resources for regular data collection and analysis.

ii) External Monitoring

Donors and international organizations will go on missions to check the status of the project or program they are financing. Such missions include discussions with various stakeholders about the status of project implementation, change in project assumptions, new project risks, or any additional information that appear to affect the project. For large scale and complex projects, donors might use a team of experts to check project accounts or certain technical, social and environment aspects.

10.5.5 Methodology/How to do a Social Impact Assessment

The different methods in each of the steps identified above, in general terms.

i) Secondary data collection and stakeholder analysis.
ii) Extensive grassroots community research to understand community concerns and identify.
iii) Assess the social impacts are both positive and negative.
iv) Identify monitoring and mitigation options.

10.5.6 What to Include in a Social Impact Assessment

i) Core SIA variables
   a. Population change
   b. Community and institutional structures
   c. Political and social resources
   d. Community and family changes
   e. Community resources
   f.

ii) Other variables
   a. Health and social wellbeing
   b. Quality of the living environment (liveability)
   c. Economic impacts and material wellbeing
   d. Cultural impacts
   e. Family and community impacts
   f. Institutional, legal, political and equity impacts
   g. Gender relations

- Appendix
  - o Sample questionnaires
  - o Sample topic guide

10.6 Reporting on the ESIA

10.6.1 Reporting on the Project Brief

As mentioned above, the environmental aspects of the project may be approved based on the project brief. A project brief should comprise of the following contents (Source: Environmental Impact Assessment Regulations, NEMA 1998):

i) Name and title, address of the developer.
ii) Name, purpose, objectives and nature of the project, including attributes such as size of the project, design, activities that shall be undertaken during and after the establishment of the project, products and inputs, sources of inputs, etc.
iii) Description of the proposed project site and its surroundings, and alternative sites, if any, where the project is to be located.
iv) Description of how the proposed project and its location conform to existing laws, regulations and policies governing such projects and the use of the site/area proposed for its location.
v) Any likely environmental impacts that may arise due to implementing various phases/ stages of the project and proposed mitigation measures thereto.
vi) Description of any other alternatives, which are being considered (*e.g.* siting, technology, construction and operation procedures, sources of raw materials, handling of wastes, etc.).
vii) Any other information that may be useful in determining the level of ESIA required.

10,6.2 Reporting on the Environmental Impact Statement (EIS)

The document prepared after carrying out the Environmental and Social Impact Assessment is called the Environmental Impact Statement (EIS). The EIS shall have the following contents:

i) Cover page – All consultants who participated in the ESIA study should be included, either on this page or in an insider page and they should sign the EIS before it is submitted to NEMA.

ii) Executive Summary: the executive summary should be a precise description of the significant results and recommended actions from the EIStudy.

iii) Project Description: the description of the proposed project should include the name/title of the project, purpose/ nature, objectives, the project worth (amount), scale and activities during the different phases of the project, including its technical, economic, social and physical contexts.

iv) Project Site: the description of the site of the proposed project should cover the surrounding areas, including the spatial and temporal boundaries within which the project is planned. The existing conditions of the physical, biological and human environment, current land-uses in the surrounding areas, as well as the trends and the anticipated future environmental conditions in the area should also be described. Environmentally sensitive areas or areas of unique bio-physical, socio-economic or cultural value should be delineated. A google map of the area or other map, which can be used to access the project site should be included. The coordinates of sampling points for water and soil samples should be taken by GPS using the Universal Transverse Mercator (UTM) system and plotted on the map.

v) Environmental Impacts: the nature and levels of the environmental impacts that are likely to result from implementing and operating the project at various stages (including adverse and beneficial impacts, long-term versus short-term impacts, unavoidable and/or irreversible significant environmental effects and growth inducing aspects), should be presented.

vi) Project Alternatives

vii) Mitigation Measures: mitigation measures should focus on achievable, pragmatic, environmentally feasible and cost-effective solutions for the various project alternatives.

viii) Monitoring and Evaluation Programme

ix) References

x) Appendices – These include Terms of Reference for EIStudy, a letter from NEMA which approved the Terms of Reference, Proof of land ownership (e.g. copies of land title deed, or sale agreement), a signed list of individuals and agencies/ organizations Consulted, and their Comments, drawings (plans and sections) of the proposed investment.

Chapter 11

PROJECT COST MANAGEMENT

11.1 Introduction

Project cost management includes the processes required to ensure that the project is completed within the approved budget. It involves resource planning, cost estimating, cost budgeting, and cost control. The costs of a water supply project to the owner include both the initial capital cost and the subsequent operation and maintenance costs. Each of these major cost categories consists of a number of cost components.

The initial capital costs for a construction project include the expenses related to the initial establishment of the facility such as:

i) Land acquisition, including assembly, holding and improvement,
ii) Planning and feasibility studies,
iii) Architectural and engineering design,
iv) Construction, including materials, equipment and labour,
v) Field supervision of construction,
vi) Construction financing,
vii) Insurance and taxes during construction,
viii) Owner's general office overhead,
ix) Equipment and furnishings not included in construction, and
x) Inspection and testing.

The operation and maintenance cost in subsequent years over the project life cycle includes the following expenses:

i) Land rent, if applicable,
ii) Operating staff,
iii) Labour and material for maintenance and repairs,
iv) Periodic renovations,
v) Insurance and taxes,
vi) Financing costs,
vii) Utilities, and
viii) Owner's other expenses.

The magnitude of each of these cost components depends on the nature, size and location of the project as well as the management organization, among many considerations. The owner is interested in achieving the lowest possible overall project cost that is consistent with its investment objectives.

It is important for design professionals and construction managers to realize that while the construction cost may be the single largest component of the capital cost, other cost components are not insignificant. For example, land acquisition costs are a major expenditure for building construction in high-density urban areas, and construction financing costs can reach the same order of magnitude as the construction cost in large projects such as the construction of nuclear power plants.

From the owner's perspective, it is equally important to estimate the corresponding operation and maintenance cost of each alternative for a proposed facility in order to analyse the life cycle costs. The large expenditures needed for facility maintenance, especially for publicly owned infrastructure, are reminders of the neglect in the past to consider fully the implications of operation and maintenance cost in the design stage.

In most construction budgets, there is an allowance for contingencies or unexpected costs occurring during construction. This contingency amount may be included within each cost item or be included in a single category of construction contingency. The amount of contingency is based on historical experience and the expected difficulty of a particular construction project.

In this chapter, we shall focus on the estimation of construction costs, with only occasional reference to other cost components.

11.2 Cost Estimation

11.2.1 Introduction

Cost estimating is one of the most important steps in project management. A cost estimate establishes the baseline of the project cost at different stages of development. A cost estimate at a given stage of project development represents a prediction provided by the cost engineer on the basis of available data. According to the American Association of Cost Engineers, cost engineering is defined as that area of engineering practice where engineering judgment and experience are utilized in the application of scientific principles and techniques to the problem of cost estimation, cost control and profitability.

Virtually all cost estimation is performed according to one or some combination of the following basic approaches:

i) Production function
ii) Empirical cost inference
iii) Unit costs of bill of quantities
iv) Allocation of joint costs

11.2.2 Production Function

In microeconomics, the relationship between the output of a process and the necessary resources is referred to as the production function. In construction, the production function may be expressed by the relationship between the volume of construction and a factor of production such as labour or capital. A production function relates the amount or volume of output to the various inputs of labour, material and equipment. For example, the amount of output Q may be derived as a function of various input factors $x_1, x_2, ..., x_n$ by means of mathematical and/or statistical methods. Thus, for a specified level of output, we may attempt to find a set of values for the input factors so as to minimize the production cost. The relationship between the size of a building project (expressed in square metres) to the input labour (expressed in labour hours per square metre) is an example of a production function for construction.

11.2.3 Empirical Cost Inference

Empirical estimation of cost functions requires statistical techniques which relate the cost of constructing or operating a facility to a few important characteristics or attributes of the system. The role of statistical inference is to estimate the best parameter values or constants in an assumed cost function. Usually, this is accomplished by means of regression analysis techniques.

11.2.4 Unit Costs for Bill of Quantities

A unit cost is assigned to each of the facility components or tasks as represented by the bill of quantities. The total cost is the summation of the products of the quantities multiplied by the corresponding unit costs. The unit cost method is straightforward in principle but quite laborious in application. The initial step is to break down or disaggregate a process into a number of tasks. Collectively, these tasks must be completed for the construction of a facility. Once these tasks are defined and quantities representing these tasks are assessed, a unit cost is assigned to each and then the total cost is determined by summing the costs incurred in each task. The level of detail in decomposing into tasks will vary considerably from one estimate to another.

11.2.4 Allocation of Joint Costs

Allocations of cost from existing accounts may be used to develop a cost function of an operation. The basic idea in this method is that each expenditure item can be assigned to particular characteristics of the operation. Ideally, the allocation of joint costs should be causally related to the category of basic costs in an allocation process. In many instances, however, a causal relationship between the allocation factor and the cost item cannot be identified or may not exist. For example, in construction projects, the accounts for basic costs may be classified according to (1) labour, (2) material, (3) construction equipment, (4) construction supervision, and (5) general office overhead. These basic costs may then be allocated proportionally to various tasks which are subdivisions of a project.

11.3 Types of Construction Cost Estimates

Construction cost constitutes only a fraction, though a substantial fraction, of the total project cost. However, it is the part of the cost under the control of the construction project manager. The required levels of accuracy of construction cost estimates vary at different stages of project development, ranging from ball park figures in the early stage to fairly reliable figures for budget control prior to construction. Since design decisions made at the beginning stage of a project life cycle are more tentative than those made at a later stage, the cost estimates made at the earlier stage are expected to be less accurate. Generally, the accuracy of a cost estimate will reflect the information available at the time of estimation.

Construction cost estimates may be viewed from different perspectives because of different institutional requirements. In spite of the many types of cost estimates used at different stages of a project, cost estimates can best be classified into three major categories according to their functions. A construction cost estimate serves one of the three basic functions: design, bid and control.

For establishing the financing of a project, either a design estimate or a bid estimate is used.

1. Design Estimates. For the owner or its designated design professionals, the types of cost estimates encountered run parallel with the planning and design as follows:
   - Screening estimates (or order of magnitude estimates).
   - Preliminary estimates (or conceptual estimates).
   - Detailed estimates (or definitive estimates).
   - Engineer's estimates based on plans and specifications.

For each of these different estimates, the amount of design information available typically increases.

2. Bid Estimates. For the contractor, a bid estimate submitted to the owner either for competitive bidding or negotiation consists of direct construction cost including field supervision, plus a mark-up to cover general overhead and profits. The direct cost of construction for bid estimates is usually derived from a combination of the following approaches.
   - Subcontractor quotations.
   - Quantity take-offs.
   - Construction procedures.

3. Control Estimates. For monitoring the project during construction, a control estimate is derived from available information to establish:
   - Budget estimate for financing.
   - Budgeted cost after contracting but prior to construction.
   - Estimated cost to completion during the progress of construction.

11.4 Design Estimates

In the planning and design stages of a project, various design estimates reflect the progress of the design. At the very early stage, the screening estimate or order of magnitude estimate is usually made before the facility is designed, and must therefore rely on the cost data of similar facilities built in the past. A preliminary estimate or conceptual estimate is based on the conceptual design of the facility at the state when the basic technologies for the design are known. The detailed estimate or definitive estimate is made when the scope of work is clearly defined and the detailed design is in progress so that the essential features of the facility are identifiable. The engineer's estimate is based on the completed plans and specifications when they are ready for the owner to solicit bids from construction contractors. In preparing these estimates, the design professional will include expected amounts for contractor's overhead and profits.

Table 11-1: Sample Preliminary Construction Cost Estimates.

Item No.	Description	Estimated Cost [UGX]
1	Intake works	
2	Raw water transmission mains	
3	Treatment works	
4	Treated water transmission mains	
5	Distribution mains	
6	Pumping station	
7	Storage reservoirs	
8	Miscellaneous works	
9	**SUB TOTAL 1**	
10	Preliminaries (15% of Sub-Total 1)	
11	**SUBTOTAL 2**	
12	Contingencies (10% of Sub-Total 2)	
13	**TOTAL**	

The costs associated with a facility may be decomposed into a hierarchy of levels that are appropriate for the purpose of cost estimation. The level of detail in decomposing the facility into tasks depends on the type of cost estimate to be prepared. For conceptual estimates, for example, the level of detail in defining tasks is quite coarse; for detailed estimates, the level of detail can be quite fine.

11-5 Bid Estimates

The contractor's bid estimates often reflect the desire of the contractor to secure the job as well as the estimating tools at its disposal. Some contractors have well established cost estimating procedures while others do not. Since only the lowest bidder will be the winner of the contract in most bidding contests, any effort devoted to cost estimating is a loss to the contractor who is not a successful bidder. Consequently, the contractor may put in the least amount of possible effort for making a cost estimate if it believes that its chance of success is not high.

If a general contractor intends to use subcontractors in the construction of a facility, it may solicit price quotations for various tasks to be subcontracted to specialty subcontractors. Thus, the general subcontractor will shift the burden of cost estimating to subcontractors. If all or part of the construction is to be undertaken by the general contractor, a bid estimate may be prepared on the basis of the quantity take-offs from the plans provided by the owner or on the basis of the construction procedures devised by the contractor for implementing the project. For example, the cost of a footing of a certain type and size may be found in commercial publications on cost data which can be used to facilitate cost estimates from quantity take-offs. However, the contractor may want to assess the actual cost of construction by considering the actual construction procedures to be used and the associated costs if the project is deemed to be different from typical designs. Hence, items such as labour, material and equipment needed to perform various tasks may be used as parameters for the cost estimates.

11.6 Control Estimates

Both the owner and the contractor must adopt some base line for cost control during the construction. For the owner, a budget estimate must be adopted early enough for planning long term financing of the facility. Consequently, the detailed estimate is often used as the budget estimate since it is sufficient definitive to reflect the project scope and is available long before the engineer's estimate. As the work progresses, the budgeted cost must be revised periodically to reflect the estimated cost to completion. A revised estimated cost is necessary either because of change orders initiated by the owner or due to unexpected cost overruns or savings.

For the contractor, the bid estimate is usually regarded as the budget estimate, which will be used for control purposes as well as for planning construction financing. The budgeted cost should also be updated periodically to reflect the estimated cost to completion as well as to insure adequate cash flows for the completion of the project.

11.7 Operation and Maintenance Costs

11.7.1 Introduction

The elements of O&M costs may include: labour; electricity; chemicals; materials; overhead; raw water charges; insurance; etc. One approach is to estimate the O&M costs as a percentage of (accumulated) investment costs. Another approach might be to analyze the utility's past performance and to relate the total O&M costs to the volume of water produced and/or distributed. And a third approach relates specific costs items to specific outputs and totals them in a second step. For example, costs of electricity and chemicals could be calculated on the basis of a specific requirement per m^3 produced and the labour requirements could be calculated on the basis of the number of employees per connection. Table 11-2 shows the economic life of the various water supply scheme components that can be used in the calculation of O& M costs. The economic life of a component is the most economic time to keep the component in service. The economic life depends on how the component is used and maintained, as well as time when more modern versions of the component become available, rendering the original version obsolete.

The Operation Costs include staff costs, energy costs, chemical costs, maintenance, and miscellaneous.

11.7.2 Staffing Costs

The main objective of operational organization is to ensure the provision of a continued and satisfactory service to the user of the water supply system at minimum cost. The staffing is usually divided into management, which provides direction, and control, operators and maintenance workers who are concerned with replacement of worn out and defective items for continuous serviceability.

The staffing for a water supply utility is likely to include engineers both civil, electrical and mechanical, chemists, supported by engineering and laboratory technicians and technical assistants, accounting, clerical and secretarial personnel. Staff numbers shall depend on the water supply system size and complexity. Additional figures for salary scales of different job groups can be found with the DWD.

11.7.3 Energy Costs

Costs of electric power are scheduled in the various tariffs of the Electricity Regulatory Authority (ERA). ERA has different methods of charging for electric power, depending on consumption. Electric power charges are made up of the following three basic components:

i) A fixed charge per reading period,
ii) A charge per unit (kilowatt hour-kWh) of electric power supplied, and
iii) A charge per kVA (kilovolt-amp) of maximum demand during the reading period.

The electricity rates are usually given per KVA. For cost estimate purposes it may be assumed that 1 KW=0.8 KVA. Diesel prices vary from one area of Uganda to another, and they can be obtained from the major fuel suppliers in the country.

11.7.4 Chemical Costs

The following dosage rates may be assumed for purposes of cost estimates, unless jar tests or other experience indicate different rates:

i) Aluminium sulphate: 0.05 kg/m3 of water.
ii) Lime: 0.002 kg/m3 of water.
iii) Soda ash: 0.02 kg/m3 of water.
iv) Tropical chloride of lime: 0.002 kg/m3 of water

Current chemical prices should be obtained from the relevant chemical suppliers.

11.7.5 Maintenance Costs

The annual maintenance costs of a water supply system can be made of the following component costs as detailed in Table 11-2 below as a percentage of the construction cost of the scheme.

Table 11-2: Annual Maintenance Costs and Economic Life.

	Component	Economic Life (Years)	Annual Maintenance Costs - % of Construction Costs
1	Dams	40	0.5
2	Intake works, including boreholes; mass concrete structures such as intakes, underground pits, culverts, etc.	40	1
3	Earthworks generally	40	1
4	Boreholes and wells	20	1
5	Pumps	10	5
6	Diesel engines	10	5
7	Electric motors, cables and switch gears	10	5
8	Piping all types	30	1
9	Treatment works: Treatment works in masonry or reinforced concrete	30	1
10	Storage tanks in masonry or reinforced concrete	30	1
11	Storage tanks: sectional steel including towers	20	2
12	Storage tanks corrugated galvanized steel (C.G.S.) on timber stands	10	2
13	Buildings C.G.S. on timber	20	1
14	Buildings, masonry	30	1

	Component	Economic Life (Years)	Annual Maintenance Costs - % of Construction Costs
15	Water kiosks, latrines, licensed retailer points etc.	20	2
16	Gantries, steelwork etc.	20	2
17	Permanent tools and plant not mentioned elsewhere	10	2
18	Water meters	10	5
19	Chemical dosing gear	10	5
20	Instruments and testing apparatus	5	5
21	Access roads	30	1
22	Fences, G.S. wire or mesh on timber	10	1
23	Fences, G.S. wire or mesh on concrete posts	20	1

11.7.6 Miscellaneous Costs

11.7.6.1 Land Acquisition

The cost of land acquisition for the scheme should be estimated in close consultation with the Chief Government Valuer and the Local Government.

11.7.6.2 Electricity Power Transmission Line

Connection to the existing national electricity grid should be estimated in conjunction with the utility supply company.

11.7.6.3 Design Costs

Only external design costs should formally be considered. The design costs for the different design stages such as Preliminary design, Feasibility design, and detailed design may vary depending on the costs of the works but may range between 8% - 10% of the project cost.

11.7.6.4 Supervision Costs

The cost of supervision is related to the cost of the works and the cost of the Resident Engineer should be assumed to be the maximum allowable salary in accordance with the MWE staff salary structure, this may be in the range of 3% – 5% of the project cost.

11.7.6.5 Supervision Costs

Establishment charges which are a contribution to the expenses of the MWE may be taken as 2% of the direct costs as described elsewhere in the manual.

11.7.6.6 Depreciation Costs

The depreciation should be assumed to be on a straight line basis and be based on the economic lifetime of the various assets and equipment in the water supply scheme.

11.7.6.7 Capital Costs

Real loan charges i.e interest and amortization should not normally be considered in the economic analysis of a particular project. Ho7wever, the annual capital cost should be estimated assuming 6a nominal interest rate. A rate of 22% in 2012 was used but this rate has to be determined from time to time depending on the ruling interest rate.

11.7.6.8 Monitoring and Evaluation Costs

The cost of monitoring and evaluation should be estimated on a 50% basis each for data collection and analysis respectively and ranges between 2% - 3% of the total project cost.

11.7.7 Life Cycle Costing (LCC)

Different project investment assets have different lifetimes and need replacement within the project lifetime. The cost of those reinvestments needs to be included in the project's benefit-cost calculation. The purpose of LCC is to estimate the overall costs of project alternatives and to select the design that ensures the facility will provide the lowest overall cost of ownership consistent with its quality and function. The LCC should be performed early in the design process while there is still a chance to refine the design to ensure a reduction in life-cycle costs (LCC).

The first and most challenging task of LCC is to determine the economic effects of alternative designs and to quantify these effects and express them in monetary terms. There are numerous costs associated with installation, operation and maintenance of water systems. These can be categorized as initial costs including design and construction costs, operation costs, maintenance cost including repairs and replacement costs and finance charges including investment/loan payments costs.

11.8 Residual Values

The residual value of project assets at the end of the project life should be included in the benefit-cost analysis as a negative cost (or benefit).

11.9 Specifications

Specification can greatly impact the construction cost. It is therefore desirable for the Ministry of Water and Environment to develop specification for water supply projects.

11.10 Standard Method of Measurement

Bills of quantities should follow a standardized format. It is desirable for the Ministry of Water and Environment to have a standard definition of bill items and standardized units of measurement.

11.11 Revenue

The revenue should be estimated using the current structure of tariff as approved by the Government of Uganda. The price may depend on location, whether the water is metered or not, type of use (domestic or industrial) and whether the water is sold to a retailer or direct to a customer. It should be assumed that 75% of the produced water is sold and paid for by consumers, 25% being leakage and waste and uncollected bills.

11.12 Presentation of Costs

The following costs should be computed and presented in all preliminary design and final design reports:

i) Investment costs for all implementation phases
ii) Investment costs per capita calculated for the initial, future, and ultimate population
iii) Investment cost per km^2 of supply area and per m^3 water of daily ultimate output
iv) Operation and maintenance costs of the initial, future and ultimate year
v) Production costs per m^3 water for initial, future and ultimate year considering only direct operation and maintenance costs

vi) Production costs per m^3 water for initial, future and ultimate year considering only direct operation, maintenance, establishment and annual capital costs.

vii) Miscellaneous costs e.g., land acquisition, connection to electricity grid, design costs, supervision costs and loan costs when relevant.

viii) Monitoring and evaluation costs estimated at 2-3% of the total project costs. This should be allocated equally to data collection and analysis.

Sample outputs from project cost estimates are presented in **Table 11-3** to **Table 11-7**.

Table 11-3 Sample Preliminary Cost Estimate

PRELIMINARY COST ESTIMATE						
Item		**Unit**	**Band**	**Quantity**	**Unit Cost**	**Total Cost (UGX)**
Surface Water Development (Lake or River)	Access Roads	km		0.8	200,000,000	160,000,000
	Reservoir (Earth Dam)	Lump sum				-
	River/lake Intake pipework, 600 mm	km	RC	0.3	500,000,000	150,000,000
		km	DI		600,000,000	-
	RW Storage/Sedimentation	No.	500 cu.m	0.5	150,000,000	75,000,000
		No.	1000 cu.m		240,000,000	-
		No.	2000 cu.m		360,000,000	-
	RW Pumps, single	No	<10 KW	2.0	25,000,000	50,000,000
	RW Power supply –	KWH	50 KVA	1.0	70,000,000	70,000,000
	RW Power Supply - Solar	No	5 KVA	2.0	60,000,000	120,000,000
		No	10 KVA		120,000,000	-
		No	15 KVA		180,000,000	-
	RW Power Supply Diesel	No	5 KVA	2.0	10,000,000	20,000,000
		No	10 KVA		15,000,000	-
		No	15 KVA		22,000,000	-
	RW Transmission Mains (DI)	Km	75 mm		25,150,000	-
		km	100mm	4.0	51,500,000	206,000,000
		km	150mm		174,150,000	-
	Alum Dosing	No		2.0	10,000,000	20,000,000
	Flocculation/Clarification	No	20 m^2	2.0	25,000,000	50,000,000
		No	30 m^2		32,000,000	-
		No	50 m^2		35,000,000	-
	Slow Sand Filtration	No.	500 m^2		300,000,000	-
		No.	1000 m^2		480,000,000	-
		No.	2000 m^2		640,000,000	-
	Rapid Sand Filtration	No.	20 m^2	2.0	65,000,000	130,000,000
		No.	30 m^2		75,000,000	-
		No.	50 m^2		85,000,000	-
	Chlorination	No		2.0	15,000,000	30,000,000
	Treated Water Storage	No.	500 cu.m	0.5	200,000,000	100,000,000
		No.	1000 cu.m		320,000,000	-
		No.	2000 cu.m		400,000,000	-
	TW Pumps, multi stage, centrifugal	No	<10 KW	2.0	25,000,000	50,000,000
		No.	10 -20 KW		35,000,000	-
		No	>20 KW		45,000,000	-
	TW Power supply –	No.	50 KVA		70,000,000	-
	TW Power Supply - Solar	No	5 KVA		60,000,000	-
		No	10 KVA		120,000,000	-
		No	15 KVA		180,000,000	-
	TW Power Supply Diesel	No	5 KVA		10,000,000	-
		No	10 KVA		15,000,000	-
		No	15 KVA		22,000,000	-
	Housing	set	Staff Housing	1.0	70,000,000	70,000,000
		m2	Plant Housing	1.0	30,000,000	30,000,000
	TW Transmission Mains (DI)	Km	100 mm	2.0	145,000,000	290,000,000
		km	150mm		170,000,000	-
		km	200mm		190,000,000	-
Town Water Reservoir	Capacity 50 cu.m	No.	Elevation 6 m		75,000,000	-
	Capacity 50 cu.m	No.	Elevation 12 m		82,500,000	-
	Capacity 100 cu.m	No.	Elevation 6 m	1.0	105,000,000	105,000,000
	Capacity 100 cu.m	No.	Elevation 12 m		114,000,000	-
	Capacity 200 cu.m	No.	Elevation 6 m		181,500,000	-
	Capacity 200 cu.m	No.	Elevation 12 m		198,000,000	-
Distribution System	UPVC / DI PN 10	km	50 mm		11,450,000	-
		Km	63mm	2.0	18,500,000	37,000,000
		km	75mm	1.0	25,150,000	25,150,000
		km	100mm		51,500,000	-
		km	150mm		74,150,000	-
	HDPE PN 10	km	63mm	2.0	26,200,000	52,400,000
		km	75mm	1.0	38,400,000	38,400,000
		Km	100mm	1.0	82,050,000	82,050,000
		Km	150mm		54,750,000	-
Public water Kiosks		No.	-	2.0	3,000,000	6,000,000
Office space		sum	-	1.0	50,000,000	50,000,000
Equipment/ tooling		SUM	No.	1.0	1,000,000	1,000,000
Surface Water (+10% contingency)	**Grand Total (Diesel)**					**2,087,800,000**
	Grand Total (Solar)					**2,120,800,000**
	Grand Total (Grid)					**2,065,800,000**

Table 11-4 Presentation of Operation and Maintenance Costs

	OPERATION AND MAINTENANCE COSTS				
	Item Description	**Units**	**Base Year (2010)**	**Intermediate Year (2020)**	**Ultimate Year (2030)**
1.0	**Water sales**				
1.1	Proportion of Population served	No.	**90%**	**92%**	**95%**
1.2	Population served	No.	9,877	15,176	23,624
1.3	Water demand	m^3/day	198	304	472
1.4	Estimated Water Sales/month	m^3/month	5,926	9,106	14,174
	Total Water Sales per month	**UGX/month**	**7,641,683**	**25,350,444**	**85,192,949**
2.0	**Energy Costs**				
2.1	**General**				
2.1.1	Pumping rate	m^3/hr	10.00	10.00	10.00
2.1.2	Daily Pump Hours run	hrs	20	30	47
2.1.3	Daily Energy Consumption (12KW)	KWh/day	237	364	567
2.2	**Diesel**				
2.2.1	Generator Fuel Consumption	L/KWH	0.30	0.30	0.30
2.2.2	Daily fuel consumption	L/day	71.11	109.27	170.09
2.2.3	Diesel Price	UGX/L	2,100.00	3,200.00	4,300.00
2.2.4	Total Energy costs (Diesel)	UGX/month	4,480,026	10,489,872	21,941,646
2.3	**Grid Power**				
2.3.1	Cost of energy	UGX/KWH	450	550	650
2.3.2	Total Energy costs (Grid)	UGX/Month	3,200,018	6,009,823	-
2.4	**Solar Power**				
2.4.1	Cost of energy	UGX/KWH	-	-	-
2.4.2	Total Energy Costs (Solar)	UGX/Month	-	-	-
	Total Energy Costs (Diesel)	**UGX/month**	**4,480,026**	**10,489,872**	**21,941,646**
	Total Energy Costs (Grid)	**UGX/month**	**3,200,018**	**6,009,823**	**-**
	Total Energy Costs (Solar)	**UGX/month**	**-**	**-**	**-**
3.0	**Asset Maintenance Costs**				
3.1	Production Wells	UGX/year	220,000	474,963	1,025,411
3.2	Pumping Stations (Pumps, Generators, CPs, Wiring)				
3.2.1	Diesel	UGX/year	4,000,000	8,635,700	18,643,829
3.2.2	Solar	UGX/year	3,800,000	8,203,915	17,711,637
3.2.3	Grid	UGX/year	700,000	1,511,247	3,262,670
3.3	Pipework (Transmission and Distribution)	UGX/year	4,811,500	10,387,668	22,426,195
3.4	Storage Tanks	UGX/year	1,050,000	2,266,871	4,894,005
3.5	Public Kiosks	UGX/year	120,000	259,071	559,315
	Total Asset Maintenance Costs (Diesel)	**UGX/month**	**850,125**	**1,835,356**	**3,962,396**
	Total Asset Maintenance Costs (Solar)	**UGX/month**	**833,458**	**1,799,374**	**3,884,714**
	Total Asset Maintenance Costs (Grid)	**UGX/month**	**575,125**	**1,241,652**	**2,680,633**

4.0	**Depreciation (economic life reciprocal)**				
4.1	Production Wells	UGX/year	550,000	1,187,409	2,563,526
4.2	Pumping Stations (Pumps, Generators, CPs, Wiring)				
4.2.1	Diesel	UGX/year	1,000,000	2,158,925	4,660,957
4.2.2	Solar	UGX/year	11,000,000	23,748,175	51,270,529
4.2.3	Grid	UGX/year	650,000	1,403,301	3,029,622
4.3	Pipework (Transmission and Distribution)	UGX/year	16,038,333	34,625,559	74,753,984
4.4	Storage Tanks	UGX/year	3,500,000	7,556,237	16,313,350
	Total Asset Depreciation Costs (Diesel)	**UGX/month**	**1,757,361**	**3,794,011**	**8,190,985**
	Total Asset Depreciation Costs (Solar)	**UGX/month**	**2,590,694**	**5,593,115**	**12,075,116**
	Total Asset Depreciation Costs(Grid)	**UGX/month**	**1,724,028**	**3,722,047**	**8,035,620**
5.0	**Chemical Costs**				
5.1	Chlorination	UGX/Month	1,884,455	2,895,642	4,507,392
6.0	**Operator Fixed Costs**	**UGX/month**	**1,600,000**	**3,454,280**	**7,457,531**
7.0	**Management Style**				
7.1	Proposed Split (Operator/Board)	No.	0.95	0.95	0.95
7.2	Management Fees(Operator)	UGX/month	7,259,599	24,082,922	80,933,302
7.3	Water Board Retained Funds	UGX/month	382,084	1,267,522	4,259,647
8.0	**Operator Surplus (Mgt Fees - Costs)**				
8.1	Surplus (Diesel)	UGX/Month	(1,555,007)	5,407,772	43,064,335
8.2	Surplus (Solar)	UGX/Month	2,941,685	15,933,626	65,083,664
8.3	Surplus (Grid)	UGX/Month	0	10,481,526	66,287,745
9.0	**Diesel**				
9.1	Management Fees (Operator)	UGX/Month	(1,477,257)	5,137,383	40,911,119
9.2	Water Board (Investment)	UGX/Month	(77,750)	270,389	2,153,217
10.0	**Solar**				
10.1	Management Fees (Operator)	UGX/Month	2,794,601	15,136,945	61,829,481
10.2	Water Board (Investment)	UGX/Month	147,084	796,681	3,254,183
11.0	**Grid**				
11.1	Management Fees (Operator)	UGX/Month	0	9,957,449	62,973,357
11.2	Water Board (Investment)	UGX/Month	0	524,076	3,314,387
12.0	**Community Financing**				
12.1	Proposed Tariff	UGX/jerry can	32	70	150

Table 11-5 Presentation of Investment Costs

Investment costs			
Water source	Energy source	Investment cost (UGX x1,000,000)	Per capita cost ($)
Groundwater	Diesel	910	46
	Solar	965	49
	Grid	976	49
Surface water	Diesel	2,088	105
	Solar	2,121	107
	Grid	2,066	104

Table 11-6 Presentation of O&M Costs

O&M costs			
Water source	Energy source	Proposed Tariff (UGX/20L)	Break even tariff (2010)
Groundwater	Diesel	50	39
	Solar	50	19
	Grid	50	32
Surface water	Diesel	50	45
	Solar	50	28
	Grid	50	38

Table 11-7 Presentation of Engineer's Estimate

WSDF-N 17 TOWNS WATER & SANITATION PROJECT					
PAIDHA TOWN WATER SUPPLY PROJECT					
BILL NOPAID/GT1: GENERAL ITEMS					
ITEM NO.	**ITEM DESCRIPTION**	**UNIT**	**QTY**	**RATE Ushs**	**AMOUNT Ushs**
	Contractual Requirements				
A110	Performance Bond				10,000,000
A130	Insurance of Construction Equipment				10,000,000
A140	Insurance against damage to persons and property				10,000,000
	Specified Requirements				
	Equipment for use by the Engineer's staff				
A230	Use of the Contractor's surveying equipment by the Project Manager				10,000,000
	Testing				
A250	Concrete works test cubes	No.	100	300,000	30,000,000
A260	Pressure Testing and Sterilisation of water mains	LS			36,000,000
A250	Allow for de-watering of works	LS			20,000,000
A420	Production of As-built drawings	LS			6,000,000
	Temporary Works				
	Provision for hoarding of site	LS			6,000,000
A279.1	Establishment and Removal of site sign boards	No.	3	2,000,000	6,000,000
A279.2	Maintenance of site-sign boards until the issue of Taking-over certificate	LS			1,500,000
	Provisional Sums				
A420.1	Alteration/relocation of existing services and structures				30,000,000
A420.2	Training of caretakers	LS			10,000,000
	TOTAL TO WATER MAINS COLLECTION				**185,500,000**

Chapter 12

FINANCIAL AND ECONOMIC ANALYSIS

12.1 Introduction

The purpose of the financial and economic analysis is to assess the financial and economic viability of the proposed project, i.e, if the proposed project is financially and economically attractive or not from the entity's viewpoint.

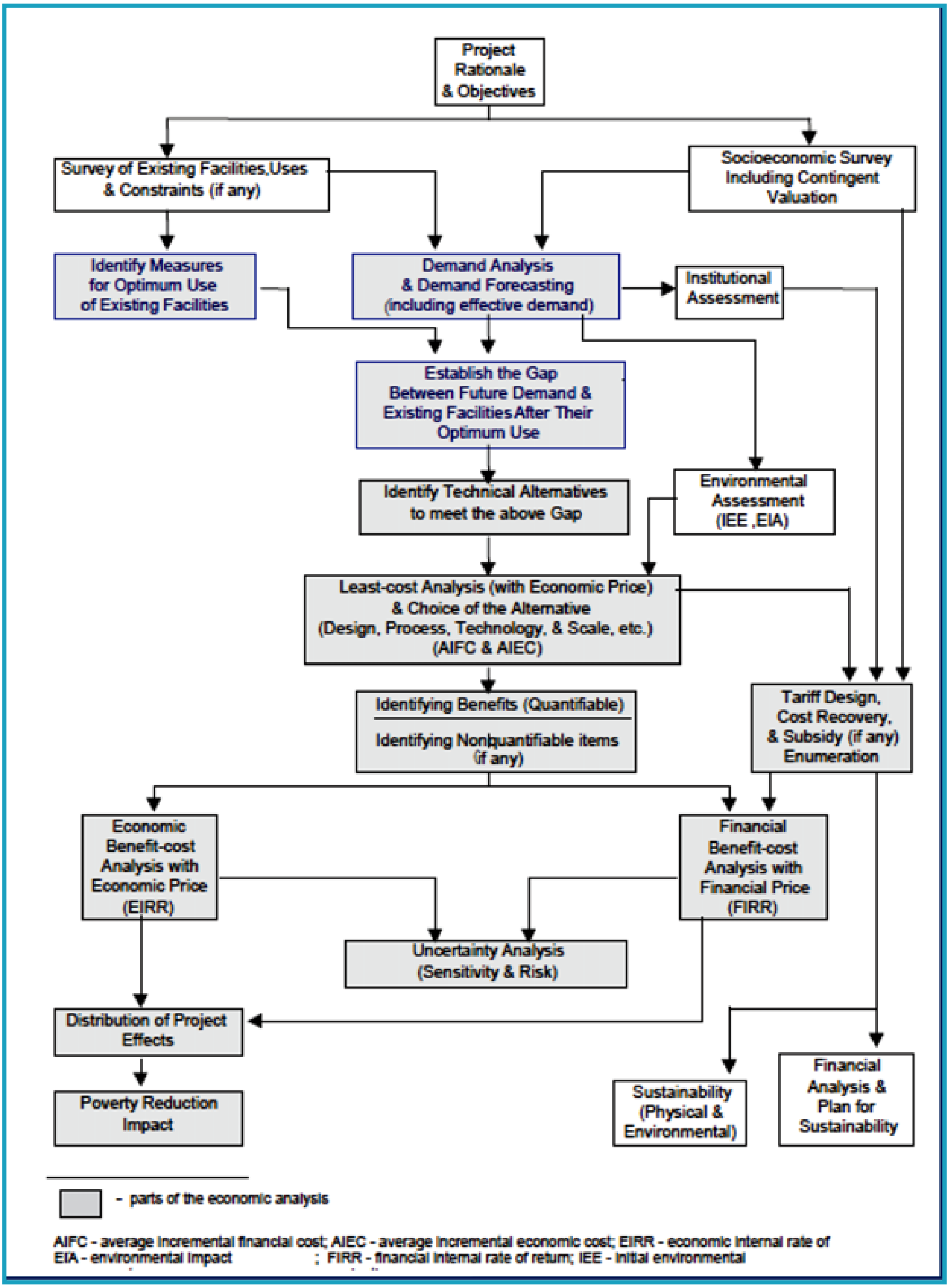
12.2 Methodology

The economic analysis of a water supply project (urban or rural) follows a sequence of interrelated steps [African Development Bank, 1998). Guidelines for the Economic Analysis of Water Supply Projects]:

i) Defining the project objectives and economic rationale;
ii) Demand analysis and forecasting effective demand for project outputs. This is to be based on either secondary information sources or socioeconomic and other surveys in the project area;
iii) Establishing the gap between future demand and supply from existing facilities after ensuring their optimum use;
iv) Identifying project alternatives to meet the above gap in terms of technology, process, scale and location through a least-cost and/or cost effectiveness analysis using economic prices for all inputs;
v) Identifying benefits, both quantifiable and non-quantifiable, and determining whether economic benefits exceed economic costs;
vi) Assessing whether the project's net benefits will be sustainable throughout the life of the project through cost-recovery, tariff and subsidy (if any) based on financial (liquidity) analysis and financial benefit-cost analysis;
vii) Testing for risks associated with the project through sensitivity and risk analyses; and
viii) Identifying and assessing distributional effects of the project and poverty reduction impact.

12.3 Timing of Financial and Economic Analysis

Economic analysis comes into play at the different stages of the project cycle: project identification, project preparation and project appraisal. At Project identification largely results from the formulation of the country sectoral strategy and country program.



Equation 12-1 Financial Analysis Flow Chart

Source: African Development Bank. "Guidelines for the Economic Analysis of Water Supply Projects", March 1998.

This means that the basic decision to allocate resources to the water supply sector for a certain (sector) project has been taken at an early stage and that the project has, in principle, been identified for implementation. In the project preparation stage, the planner has to make an optimal choice of the design, process, technology, scale and location etc. based on the most efficient use of the countries' resources.

In the project appraisal stage, the economic analysis plays a substantial part to ensure optimal allocation of a nation's resources and to meet the sustainability criteria set by both the recipient country from the social, institutional, environmental, economic and financial viewpoints.

12.4 Financial Vs. Economic Analysis

Financial and economic analyses have similar features. Both estimate the net benefits of an investment project based on the difference between the with-project and the without-project situations. However, the concept of financial net benefit is not the same as economic net benefit. While financial net benefit provides a measure of the commercial (financial) viability of the project on the project-operating entity, economic net benefit indicates the real worth of a project to the country.

Financial and economic analyses are also complementary. For a project to be economically viable, it must be financially sustainable. If a project is not financially sustainable, there will be no adequate funds to properly operate, maintain and replace assets; thus the quality of the water service will deteriorate, eventually affecting demand and the realization of financial revenues and economic benefits.

It has sometimes been suggested that financial viability not be made a concern because as long as a project is economically sound, it can be supported through government subsidies. However, in most cases, governments face severe budgetary constraints and consequently, the affected project entity may run into severe liquidity problems, thereby jeopardizing even its economic viability.

The basic difference between the financial and economic benefit-cost analyses of the project is that the former compares benefits and costs to the enterprise in constant financial prices, while the latter compares the benefits and costs to the whole economy measured in constant economic prices. Financial prices are market prices of goods and services that include the effects of government intervention and distortions in the market structure. Economic prices reflect the true cost and value to the economy of goods and services after adjustment for the effects of government intervention and distortions in the market structure through shadow pricing of the financial prices. In such analyses, depreciation charges, sunk costs and expected changes in the general price should not be included.

In financial analysis, the taxes and subsidies included in the price of goods and services are integral parts of financial prices, but they are treated differently in economic analysis. Financial and economic analyses also differ in their treatment of external effects (benefits and costs), favourable effects on health and the unaccounted-for-water of a water supply project. Economic analysis attempts to value such externalities, health effects and nontechnical losses.

12.5 Financial Vs. Economic Viability

The steps in determining the financial viability of the proposed project include:

i) Identifying and quantifying the costs and revenues;
ii) Calculating the project net benefits;
iii) Estimating the average incremental financial cost, Financial Net Present Value and Financial Internal Rate of Return (FIRR).

The FIRR is the rate of return at which the present value of the stream of incremental net flows in financial prices is zero. If the FIRR is equal to or greater than the financial opportunity cost of capital, the project is considered financially viable. Thus, financial benefit-cost analysis covers the profitability aspect of the project.

The steps in determining the economic viability of a project include the following:

i) Identifying and quantifying (in physical terms) the costs and benefits;
ii) Valuing the costs and benefits, to the extent feasible, in monetary terms; and
iii) Estimating the EIRR or economic net present value (NPV) discounted at economic opportunity cost of capital (EOCC)

The EIRR is the rate of return for which the present value of the net benefit stream becomes zero, or at which the present value of the benefit stream is equal to the present value of the cost stream. For a project to be acceptable, the EIRR should be greater than the economic opportunity cost of capital. The Government of Uganda uses 12 per cent as the minimum rate of return for projects; but for projects with considerable non-quantifiable benefits, 10 per cent may be acceptable.

In economic analysis, the market prices of inputs and outputs are adjusted to account for the effects of government intervention and market structure. The adjusted prices are termed as shadow prices and are based either on the supply price, the demand price, or a weighted average of the two. Different shadow prices are used for incremental output, non-incremental output, incremental input and non-incremental input.

Non-incremental outputs are project outputs that replace existing water production or supply. For example, a water supply project may replace existing supply by water vendors or household/community wells. Incremental outputs are project outputs that add to existing supply to meet new demands. For example, the demand for water is expected to increase in the case of a real decline in water supply costs or tariffs. Incremental inputs are for project demands that are met by an overall expansion of the water supply system. Nonincremental inputs are inputs that are not met by an expansion of overall supply but from existing supplies, i.e., taking supply away from existing users. For example, water supply to a new industrial plant is done by using water away from existing agricultural water.

12.6 Methods of Economic Analysis

12.6.1 Introduction

In order to choose the best capital investment for the firm, management requires that a number of available alternative investments be evaluated using an appropriate method of comparison. Since capital projects have different terms of costs, benefits, and timing, the basis of comparison must take into account these differences as well as the time value of money. There are several methods used for evaluating investments in engineering economy studies: net present value, equivalent annual worth, internal rate of return, External rate of return, profitability index, benefit/cost ratio, payback period, cost effective methods, capital recovery with return and capitalized equivalent. These methods are described in this Chapter.

12.6.1 2 Net Present Value [NPV]

The net present value method requires that all cash flows be discounted to their present value, using the firms required rate of return. It can be expressed as:

$$NPV = \sum_{t=0}^{n} \frac{A_t}{(1+i)^t} - C_0$$ **Equation 12-2**

Where:

A_t = cash flow for period t
C_0 = initial cost of the project
t = time
n = no. of years
i = interest rate

For a given project, once the cash flows have been discounted to present dollars, they can be compared to the costs of the projects, usually measured in present dollars. If the present value of the cash flows is equal to or greater than the cost of the investment, the project is profitable and should be accepted. The project with a larger NPV is considered a better investment. The required rate of return is the rate the firm can be obtained from comparable investment alternatives. It is also known as the discount rate, the cost of capital, the hurdle rate, or the minimum acceptable rate of return. The advantaged of the net present value method are that it takes into account the time value of money and regardless of the pattern of cash flows; a single net present value is easily calculated.

When the cost of the project is spread over a number of years, the net present value is obtained as follows:

$$NPV = \sum_{t=0}^{n} \frac{A_t}{(1+i)^t} - \sum_{t=0}^{n} \frac{C_t}{(1+i)^t}$$ **Equation 12-3**

A slight variation to this method is called the present worth method. This method is used by considering either costs alone, benefits alone or costs and benefits together. It discounts all future sums to the present, using an appropriate discount rate.

12.6.1 3 Equivalent Annual Worth [EAW]

Frequently, we wish to compare alternative cash flows in terms of uniform annuities. This method expresses all future and past costs and revenues in terms of year-end uniform payment [A]

$$EAW_j = AW[B_j] - AW[C_j]$$ **Equation 12-4**

The advantage of the Equivalent Annual Worth method is in resolving the dilemma faced by a firm which must choose between two investments alternatives of different useful lives. The NPV and EAW methods are mainly used in private sector projects.

12.6.1 4 Internal Rate of Return [IRR]

This is the method most widely used by engineers and business managers in evaluating capital projects. This method expresses profitability of a capital investment in percentage terms, a measure that is easily understood by experts and laymen alike. The internal rate of return for an investment is the rate of return [i.e. an interest rate] that makes the present value of the cash flow equal to the cost of the investment. Mathematically, it can be calculated from

$$C_0 = \sum_{t=0}^{n} \left[\frac{A_t}{(1+r)^t} \right]$$ **Equation 12-5**

Where:

C_0 = initial capital outlay
A_t = cash flow in period t
R = internal rate of return, IRR of the investment

IRR of an investment is the discount rate that makes the NPV of the investment equal to zero.

When the cash flow are not uniform series, the IRR must be calculated by trial and error or using a computer program. When the IRR of the investment is greater than the cost of capital of the firm, the investment is accepted. Sometimes the IRR must exceed the so-called required rate of return of the project or the firms minimum acceptable rate of return.

The IRR method is plagued by two main concerns:

a) Revenues from the project are assumed to be reinvested at the internal rate. If this rate is high or low relative to Minimum Attractive Rate of Return [market rate, MARR], this assumption is unrealistic. The assumption concerning re-investment of incomes can bias the selection of projects depending on whether net incomes are realised either early or late on the time line.
b) For complex cash flows [e.g., those associated with large resource developments] where cash flow fluctuates between positive and negative we can obtain multiple internal rates.

These two problems can be solved by:

a) Plotting the NPV versus i% and juxtaposing MARR on the plot. If NPV Is positive we invest, if negative we do not invest.

b) Use the external rate of return [ERR]

12.6.5 External Rate of Return [ERR]

The ERR method returns an i% similar to the IRR method, however, assumptions concerning re-investment of incomes from a project investment are made at MARR. This is more realistic than IRR. Also since points of inflection between positive and negative cash flows are adjusted, multiple rates are not obtained. There are two methods for obtaining the external rate of return from an investment.

12.6.6 Precise Method

Steps:

i) Obtain point or points of trade-off between positive and negative regions of cash flow. This is referred to the point of balance on the time line [may be somewhat subjective, may vary for complex cash flows];
ii) Discount all revenues forward to the point of balance at MARR;
iii) Discount all costs forward to point of balance at the known external rate i%;
iv) Discount all future revenues and costs back to the point of balance at i%; and
v) Equate the revenues and costs and solve for the external rate.

12.6.7 Approximate Method

Steps:

i) Discount all revenues forward to point of last cash flow at MARR;
ii) Discount all costs forward to point of last cash flow at unknown rate i%; and
iii) Equate all discounted revenues and costs and solve for i%.

12.6.8 Benefit/Cost Ratio Analysis [B/C]

12.6.8.1 Introduction

The B/C ratio method discounts all benefits and costs to a common point in time [normally the present, t = 0] and expresses the result as a ratio, such that:

$$B/C_j = \frac{PW(B_j)}{PW(C_j)} \qquad \textbf{Equation 12-6}$$

Where PW (B_j) present worth of benefits at time j
PW (C_j)present worth of benefits at time j
AW Annual worth

AW may be used instead of PW. This method is also called the Profitability Index Method. This method gives the same result as the NPV method. When faced with two alternatives, the B/C can be misleading. It is required to calculate the B/C ratio on incremental cost of the project. If greater than 1.0 we take that project to be more profitable.

12.6.8.2 Absolute Approach

All alternatives are compared to a base do nothing alternative and decision is made on choice or rank alternative relative to base. This method is used to rank non-mutually exclusive alternatives.

Steps:

i) Arrange all alternatives in order of increasing capital [or operating costs]
ii) Select base alternative usually lowest capital cost option [lowest benefits]
iii) Select method [e.g. B/C ratio] and compare to base
iv) Select alternative with highest B/C ratio [or highest NPW, EAW, etc.]

12.6.8.3 Incremental Approach

This method involves a step-wise procedure for comparing alternatives two at a time.

Arrange alternatives in order of increasing capital or operating costs.

Base alternative [A1]: Select A1 as basis. Compare A2 to A1. Select A2 or A1. If A1, delete A2. Compare A3 to A1. Select A3 or A1. If A3, compare A4 to A3.

12.6.8.4 Criteria for Selection

NPW, highest cost alternative if NPW>0; B/C ratio higher cost alternative if B/C>1.0; IRR higher cost alternative if i_R>MARR. This method yields alternative whose extra benefit per $ or extra investment is greater than zero. On lower layers of investment, the return can actually be higher than on the latter layers [higher levels of investment]. This is not the same as highest benefit to cost ratio.

12.6.9 Payback Period Method

12.6.9.1 Introduction

Sometimes firms are concerned with the number of years required to recover the initial outlay of an investment. Payback period refers to the number of years (months etc.) at which a project investment begins to pay for itself. The payback period is used to evaluate the feasibility of projects in such cases.

Presumably the shorter the payback period the better is the investment. Payback period is found in two ways; conventionally and by discounting the cash flows.

12.6.9.2 Conventional Payback

The payback period is simply obtained by counting the number of years it takes for cumulative cash flows to equal the initial investment.

12.6.9.3 Discounted Payback

This method requires that the cash flows be discounted using the required rate of return, before they are added up to equal the initial investment.

The payback method has two basic problems when it is used to compare alternatives.

a) future benefits after payback period are ignored; and
b) for the undiscounted method, future revenues are treated the same regardless of when they occur in the future. This ignores opportunity cost of capital over time.

12.6.9.4 Cost-Effective Methods

This method is useful to compare alternatives where significant, non-monetary outputs are involved. The benefits are expressed in subjective terms, for example some aesthetics ratings, some comfort index, some serviceability index and so on. This method is normally supplemented by other methods. The modification to this method is known as rank-best expected value method. In this method project alternatives are ranked as a function of their ability to meet the standard set for each objective. In addition each objective is given a weight or rank according to its order of importance. The most desirable alternative is given the highest ranking score.

12.6.9.5 Capital Recovery Cost with Return

When a capital asset such as a machine or a computer is purchased, it is expected that its use will generate enough income to recover the original cost of the investment.

$$CR = P[A/P, i, n]$$ **Equation 12-7**

Where:

P = Initial cost of the asset
n = Estimated service life in years
CR = Capital Recovery cost per year.

If salvage value is included the formula becomes:

$$CR = P[A/P, i, n] - F[A/F, i, n]$$ **Equation 12-8**

Where:

F = Forecasted salvage value of the asset at the end of year n and
CR = Capital recovery with return per year.

Given two or more alternatives, the decision rule, using the capital recovery method, is that the investment with the smallest value of CR is accepted.

12.6.9.6 Capitalized Equivalent

Financing public projects and setting up endowments usually involves the determination of the amount of investment required at the present time to generate annual cash flows in perpetuity.

$$C_o = A[P/A, i, \infty]$$ **Equation 12-9**

Where C_o = present value of the investment

This equation reduces to:

$$CE = A/i$$ **Equation 12-10**

Where CE = Capital Equivalent

The rule is that for a given rate of return the investment that yields the highest annual cash flow should be accepted.

12.6.10 Treatment of Unequal Lives

Treatment of unequal lives can be calculated using the methods below

i) Lowest common multiple life method: All alternative cash flows are repeated to coincide with lowest common life span.
ii) Use shortest life span and apply appropriate salvage values to longer life alternatives
iii) Extend value of shorter life assets to reflect the longer life span Equivalent annual work method.

12.6.11 Example 1

Calculate the payback period for the following three projects. Assuming the initial outlay and their estimated cash flows are as indicated in Table 12-1. What is the best project and what is the raking of the others?

Table 12-1 Comparison of Payback Period of 3 projects

Year	Project A	Project B	Project C
0	-2400	-2400	-2400
1	600	800	500
2	600	800	700
3	600	800	900
4	600	800	1100
5	600	800	1300

Solution

With no discounting, the best project is B [payback period = 3 years] followed by C [3.27 years] and the A [4 years].With discounting, the best project is B [payback period = 3.75 yrs] followed by C [3.92 years] and the A [5 years, must be rejected as payback period exceeds life of investment].Using NPV: The best project is C, followed by B, then A [Project A will be rejected because it has a negative cash flow]. Note that these methods gave different results.

12.6.12 Example 2

Given two projects below, use the B/C ration and NPV to find the best alternative project.

Table 12-2 Selection of alternative project using B/C and NPV methods

	Initial Outlay [$]	Discounted Cash Flow [$]
Project A	5,000	7,582
Project B	20,000	24,072

Solution

Project	Initial Outlay [$]	Discounted Cash Flow [$]	B/C	NPV [$]
Project A	5,000	7,582	1.52	2,582
Project B	20,000	24,072	1.2	4,072
Incremental	15,000	16,490	1.10	

In conclusion, Project A performed better based on both B/C and NPV analysis

12.7 Comparing Alternative Investments

12.7.1 Introduction

A systematic evaluation of each proposal and then a basis for comparing the various alternatives is required to obtain the best investment. The typical firm is faced with many capital investment opportunities each year. Theoretically, the firm should allocate capital for all proposals with expected cash flows that promise to increase the value of the shareholder's equity. In reality there are many constraints that limit the firm's ability to select all the investment opportunities available. These include:

a) Conflicting proposals
b) Limited capital resources

12.7.2 Types of Alternatives

In analysing investment proposals, it is necessary to identify all alternatives and classify them as follows:

a) Independent
b) Dependent
c) Mutually exclusive

An alternative is defined as one option, out of several, for an investment proposal under consideration.

Independent alternatives are investment alternatives such that when an alternative is selected from a group, the other alternatives are not affected by the selection. The following are examples of independent alternatives:

- Air conditioner, [Cost = $4,000, NPV = $485]
- Desktop Computer, [Cost = $3,000, NPV = $260]
- Fork-lift Truck, [Cost = $5,000, NPV = $275]
- Photographic Equipment, [Cost = $6,500, NPV = $-750]

Assuming that the firm has the funds to accept any alternative investments with positive net present value, the first three of the above alternatives are acceptable. Notice that the choice of the air-conditioner does not affect the other alternatives.

Dependent alternatives: Some of the selection of one alternative is contingent [dependent, conditional, reliant, subject] on the selection of another.

Mutually exclusive alternatives: Frequently the acceptance of one alternative automatically eliminates all the others.

12.7.3 Comparing Alternatives

The choice of method depends on complexity of the data. In general, surveys indicate that the preferred methods are IRR and NPV. Although most managers favor the IRR, economists believe that the NPV is the best method.

In evaluating independent project, both the IRR and the NPV methods give the same results. However when mutually exclusive projects are compared a conflict can occur between IRR and NPV.

The working capital, different outlays, different timing of cash flows, different lives, all affect choice of the best alternative.

12.7.4 Example

A company is considering the purchase of two machines in next year's budget. Machine A will cost $115,000 and machine B will cost $100,000. The forecasted earnings are shown in Table 12-3:

Table 12-3 Comparison of forecasted earnings of 2 machines

Year	1	2	3	4	5
Machine A [$]	32,000	34,000	36,000	38,000	40,000
Machine B [$]	30,000	31,000	32,000	33,000	34,000

The firm uses the straight-line method of depreciation for 5-year property, its cost of capital is 10%, and its marginal tax rate is 40%.

Solution: Using NPV

Machine A	1	2	3	4	5		
Revenue [$]	32,000	34,000	36,000	38,000	40,000		
Depreciation [straight line]	23,000	23,000	23,000	23,000	23,000		
Income before Tax [$]	9,000	11,000	13,000	15,000	17,000		
Tax [40%]	3,600	4,400	5,200	6,000	6,800		
Net Income {$]	5,400	6,600	7,800	9,000	10,200		
Cash Flow [$]	28,400	29,600	30,800	32,000	33,200		
P/F, 10%, n	0.9091	0.8264	0.7513	0.6830	0.6209		
NPV	**25,818**	**24,463**	**23,140**	**21,856**	**20,615**	=	115,893
						Initial Cost	115,000
Interest Rate	10%					NPV	893

Machine B	1	2	3	4	5		
Revenue [$]	30,000	31,000	32,000	33,000	34,000		
Depreciation [straight line]	20,000	20,000	20,000	20,000	20,000		
Income before Tax [$]	10,000	11,000	12,000	13,000	14,000		
Tax [40%]	4,000	4,400	4,800	5,200	5,600		
Net Income {$]	6,000	6,600	7,200	7,800	8,400		
Cash Flow [$]	26,000	26,600	27,200	27,800	28,400		
P/F, 10%, n	0.9091	0.8264	0.7513	0.6830	0.6209		
NPV	**23,636**	**21,983**	**20,436**	**18,988**	**17,634**	=	102,678
						Initial Cost	100,000
Interest Rate	10%					**NPV**	**2,678**

Choose Machine B

Machine A	1	2	3	4	5		
Revenue [$]	32,000	34,000	36,000	38,000	40,000		
Depreciation [straight line]	23,000	23,000	23,000	23,000	23,000		
Income before Tax [$]	9,000	11,000	13,000	15,000	17,000		
Tax [40%]	3,600	4,400	5,200	6,000	6,800		
Net Income {$]	5,400	6,600	7,800	9,000	10,200		
Cash Flow [$]	28,400	29,600	30,800	32,000	33,200		
P/F, 10%, n	0.9067	0.8220	0.7453	0.6757	0.6127		
NPV	**25,749**	**24,332**	**22,955**	**21,623**	**20,340**	=	115,000
						Initial Cost	115,000
Interest Rate	10.30%					NPV	0

Machine B	1	2	3	4	5		
Revenue [$]	30,000	31,000	32,000	33,000	34,000		
Depreciation [straight line]	20,000	20,000	20,000	20,000	20,000		
Income before Tax [$]	10,000	11,000	12,000	13,000	14,000		
Tax [40%]	4,000	4,400	4,800	5,200	5,600		
Net Income {$]	6,000	6,600	7,200	7,800	8,400		
Cash Flow [$]	26,000	26,600	27,200	27,800	28,400		
P/F, 10%, n	0.9007	0.8112	0.7307	0.6581	0.5927		
NPV	**23,418**	**21,579**	**19,874**	**18,295**	**16,834**	=	100,000
						Initial Cost	100,000
Interest Rate	11.03%					**NPV**	**0**

Choose machine B

I12.7 dentification and Quantification of Costs

In estimating the economic costs, some items of the financial costs are to be excluded while other items, which are not part of financial costs, are to be included. The underlying principle is that project costs represent the difference in costs between the without-project and the with-project situations. Cost items and the way they are to be treated in project economic analysis, are as follows:

i) Sunk Costs. They exist in both with-project and without-project situations and thus are not additional costs for achieving benefits. They are, therefore, not to be included.
ii) Contingencies. As the economic benefit-cost analysis is to be done in constant (or real) prices, the general price contingencies should not be included.
iii) Working Capital. Only inventories that constitute real claims on the nation's resources should be included in the project economic costs. Others items of working capital reflect loan receipts and repayment flows are not to be included.
iv) Transfer payments. Taxes, duties and subsidies are transfer payments as they transfer command over resources from one party (taxpayers and subsidy receivers) to another (the government, the tax receivers and subsidy givers) without reducing or increasing the amount of resources available in the economy as a whole. Hence, these transfer payments are not economic costs. However, in certain circumstances when valuing the economic cost of an input or an output, taxes are to be included:
    a) If the government is correcting for external environmental costs by a correcting tax to reduce the production of water, such a transfer payment is part of the economic costs.
    b) The economic value of incremental outputs will include any tax element imposed on the output, which is included in the market price at which it sells.
v) External Costs. Environmental costs arising out of a project activity, such as river water pollution due to discharge of untreated sewage effluent, is an instance of such costs. It may be necessary to internalize this external cost by including all relevant effects and investments like pollution control equipment costs and effects in the project statement.
vi) Opportunity Cost of Water. If, for example, a drinking water project uses raw water diverted from agriculture, the use of this water for drinking will result in a loss for farmers. These costs are measured as the opportunity cost of water which, in this example, equals the "benefits foregone" of the use of that water in agriculture.
vii) Depletion Premium. In water supply projects where the source of water is ground water and the natural rate of recharge or replenishment of the aquifer is less than its consumptive use, the phenomenon of depletion occurs. In such cases, significant cost increase may take place as the aquifer stock depletes; the appropriate valuation of water has to include a depletion premium in the economic analysis.
viii) Depreciation. The stream of investment assets includes initial investments and replacements during the project's life. This stream of expenditure which is included in benefit-cost analysis will generally not coincide with the time profile of depreciation and amortization in the financial accounts and as such, the latter should not be included once the former is included.

12.8 Identification and Quantification of Benefits

The gross benefit from a new water supply is made up of two parts:

i) Resource cost savings on the non-incremental water consumed in switching from alternative supplies to the new water supply system resulting from the project; and
ii) The Water Treatment Plant for incremental water consumed.

Resource cost savings are estimated by multiplying the quantity of water consumed without the project (i.e., non-incremental quantity) by the average economic supply price in the without-project situation.

The WTP for incremental supplies can be estimated through a demand curve indicating the different quantities of water demand that could be consumed at different price levels between the without-project level of demand and the with-project level of demand. The economic value of incremental consumption is the average value derived from the curve times the quantity of incremental water. Where there is inadequate data to estimate a demand curve, a contingent valuation methodology can be applied to obtain an estimate of WTP for incremental output.

The gross benefit stream should be adjusted for the economic value of water that is consumed but not paid for, i.e., sold but not paid for (bad debts) and consumed but not sold (non-technical losses). It can be assumed that this group of consumers derives, on the average, the same benefit from the water as those who pay.

Other benefits of a water supply project P include health benefits. These benefits are due to the provision of safe water and are also likely to occur provided that the adverse health impacts of an increased volume of wastewaters can be minimized.

12.9 Valuation of Economic Costs and Benefits

The economic costs and benefits must be valued at their economic prices. For this purpose, the market prices should be converted into their economic prices to take into account the effects of government interventions and market structures. The economic pricing can be conducted in two different currencies (national vs. foreign currency) and at the two different price levels (domestic vs. world prices).

To remove the market distortions in financial prices of goods and services and to arrive at the economic prices, a set of ratios between the economic price value and the financial price value for project inputs and outputs are used to convert the constant price financial values of project benefits and costs into economic values. These are called conversion factors, which can be used for groups of typical items, like energy and water resources.

12.10 Sensitivity and Risk Analysis

12.10.1 Introduction

The financial and economic benefit-cost analysis of water supply projects (WSPs) is based on forecasts of quantifiable variables such as demand, costs, water availability and benefits. The values of these variables are estimated based on the most probable forecasts, which cover a long period of time. The values of these variables for the most probable outcome scenario are influenced by a great number of factors, and the actual values may differ considerably from the forecasted values, depending on future developments. It is therefore useful to consider the effects of likely changes in the key variables on the viability (EIRR and FIRR) of a project. Performing sensitivity and risk analysis does this. Sensitivity Analysis shows to what extent the viability of a project is influenced by variations in major quantifiable variables. Risk Analysis considers the probability that changes in major quantifiable variables will actually occur.

The viability of projects is evaluated based on a comparison of its internal rate of return (FIRR and EIRR) to the financial or economic opportunity cost of capital. Alternatively, the project is considered to be viable when the Net Present Value (NPV) is positive, using the selected EOCC or FOCC as discount rate. Sensitivity and risk analyses, therefore, focus on analyzing the effects of changes in key variables on the project's IRR or NPV, the two most widely used measures of project worth.

In the economic analysis of WSPs, there are also other aspects of project feasibility which may require sensitivity and risk analysis. These include:

i) Demand Analysis: to assess the sensitivity of the demand forecast to changes in population growth, per capita consumption, water tariffs, etc.
ii) Least Cost Analysis: to verify whether the selected least-cost alternative remains the preferred option under adverse conditions;
iii) Sustainability Analysis: to assess possible threats to the sustainability of the project.
iv) Distributional Analysis: to analyze whether the project will actually benefit the poor.

Sensitivity and risk analyses are particularly concerned with factors, and combinations of factors, that may lead to unfavourable consequences. These factors would normally have been identified in the project (logical) framework as “project risks” or “project assumptions”. Sensitivity analysis tries to estimate the effect on achieving project objectives if certain assumptions do not, or only partly, materialize. Risk analysis assesses the actual risk that certain assumptions do not, or partly, occur.

Table 12-4: Variables in Water Supply Projects

Variables in Water Supply Projects to be considered in Sensitivity Analysis		
Possible Key Variables	Quantifiable Variables	Underlying Variables
Water Demand	Population growth Achieved coverage Household Consumption Non Domestic Consumption Unaccounted for Water	· Price Elasticity · Income Elasticity
Investment Costs (Economic & Financial)	Water Demand Construction Period Real Prices Conversion Factors	
O&M Costs	Personnel Costs (wages/No. of staff, etc.) Cost of Energy Cost of Maintenance Efficiency of Utility	
Financial Revenues	Quantity of water consumed Service level Income from connection fees	· Water Tariffs · UFW (bad debts)
Economic Benefits	Water Demand Resource Costs Savings	· Willingness to Pay
Cost Recovery	Water Tariffs Subsidies	

Table 12-5: Sensitivity Analysis - Example

Item	Change	NPV	IRR %	SI (NPV)	SV (NPV)
Base Case		126	13.7		
Investment	+ 20%	- 211	9.6	13.3	7.5%
Benefits	- 20%	-294	7.8	16.6	6%
O&M Costs	+ 20%	68	12.9	2.3	43.4%
Construction delays	one year	-99	10.8		

SI = Sensitivity Indicator, SV = Switching Value

In case of an increase in investment costs of 20 per cent, the sensitivity indicator is 13.3. This means that the change of 20 per cent in the variable (investment cost) results in a change of (13.3 x 20 per cent) = 266 per cent in the ENPV. It follows that the higher the SI, the more sensitive NPV is to change in the concerned variable.

In the same example, the switching value is 7.5 per cent which is the reciprocal value of the SI x 100. This means that a change (increase) of 7.5 per cent in the key variable (investment cost) will cause the ENPV to become zero. The lower the SV, the more sensitive NPV is to change in the variable concerned and the higher the risk with the project.

At this point the results of the sensitivity analysis should be reviewed. It should be asked: (I) which are the variables with high sensitivity indicators; and (ii) how likely are the (adverse) changes (as indicated by the switching value) in the values of the variables that would alter the project decision?

12.11.2 Measuring the Risk of a Project

When there is uncertainty about the returns of a project, several cash flows may be forecasted, of for each probable outcome. The expected cash flow and its variance are calculated using the traditional formulas of expected value and standard deviation of probability distributions from mathematical statistics.

The expected value of cash flow is obtained from:

$$\overline{CF} = \sum_{i=1}^{n} P_i CF_i \qquad \textbf{Equation 12-11}$$

Where CF is a forecasted cash flow, P is its probability, and n is the number of forecasts.

The standard deviation of the expected cash flow is computed from:

$$\sigma = \sqrt{\sum_{i=1}^{n} P_i (CF_i - \overline{CF}_i)^2} \qquad \textbf{Equation 12-12}$$

The standard deviation is the weighted average of the variance of the forecasted cash flows from the mean or expected cash flow. It is used to measure the relative risk of the project.

When the expected cash flows of the alternatives being compared are significantly different, a more precise measure of relative risk is the coefficient of variation, obtained as follows:

$$CV = \frac{\sigma}{\overline{CF}} \qquad \textbf{Equation 12-13}$$

The higher the coefficient of variation, the higher the risk of the project

12.11.3 Project NPV

Once the expected cash flow of a set of outcomes has been calculated, the NPV of the project can be determined, using the following:

$$NPV = \overline{CF}\,(P/A, i, n) - \qquad \textbf{Equation 12-14}$$

Where i is the cost of capital adjusted for risk.

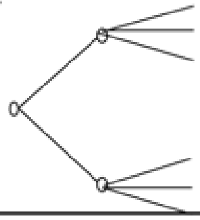
The most frequently used method to adjusting for risk is to add a premium to the cost of capital.

$$NPV = \sum_{i=1}^{n} \frac{CF_t}{(1+i+r)^t} - C_0 \qquad \textbf{Equation 12-15}$$

Where (i+r) is the risk –adjusted discount rate.

12.11.4 Decision Tree Analysis

This method involves identifying the possible outcomes of two or more alternatives in sequence, where one activity makes it possible for a set of outcomes to occur subject to a probability distribution. The objective is to find the expected value of all probable outcomes.



Action	Demand Probability	NPV	Branch	Expected NPV	Standard Deviation	CV
Small Plant	0.35	800,000	1	350,000	384,057	1.10
	0.40	300,000	2			
	0.25	- 200,000	3			
	0.35	2,000,000	4	750,000	1,013,657	1.35
Big Plant	0.40	500,000	5			
	0.25	- 600,000	6			

12.11.5 Example

1. A company plans to introduce two new products next year, each having a net capital outlay of $1,000,000. Market research indicates that the cash flows of the products will be influenced by the state of the economy. Based on economic projections, four possible levels of economic growth and their associated probabilities are forecasted for the next 3 years. Given these probabilities of the state of the economy, the firm has forecasted annual cash flows for each product as follows:

State of the Economy	Probability	Cash Flow for Product A	Cash Flow for Product B
High Growth	20%	700,000	600,000
Normal Growth	40%	500,000	450,000
Low Growth	25%	400,000	440,000
Recession	15%	200,000	400,000

Calculate each product's expected cash flow and its variance.

Solution

State of the Economy	Probability	Cash Flow for Product A	Cash Flow for Product B
High Growth	20%	700,000	600,000
Normal Growth	40%	500,000	450,000
Low Growth	25%	400,000	440,000
Recession	15%	200,000	400,000
	Expected Value	470,000	470,000
	Standard Deviation	151,987	67,082

The expected cash flow is $470,000 for both products, the standard deviation is $152,000 for A and $67,100 for B. Product A has a relatively higher risk than product B.

2. Consider the following two projects. Small Plant costs $1,200,000 and the big plant costs $3,000,000.

Market Demand	Probability	NPV Small Plant [$]	NPV Large Plant [$]
High	0.35	2,000,000	5,000,000
Normal	0.4	1,500,000	3,500,000
Low	0.25	1,000,000	2,400,000

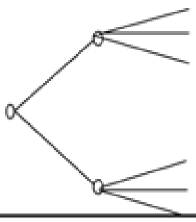
a) Draw a decision tree diagram.
b) Calculate the expected NPV of each alternative.
c) What is the relative risk of the two investments?

Solution

Market Demand	Probability	NPV Small Plant [$]	NPV
High	0.35	2,000,000	800,000
Normal	0.4	1,500,000	300,000
Low	0.25	1,000,000 -	200,000

Market Demand	Probability	NPV Large Plant [$]	NPV
High	0.35	5,000,000	2,000,000
Normal	0.4	3,500,000	500,000
Low	0.25	2,400,000 -	600,000

Action	Demand Probability	NPV	Branch	Expected NPV	Standard Deviation	CV
Small Plant	0.35	800,000	1	350,000	384,057	1.10
	0.40	300,000	2			
	0.25	- 200,000	3			
	0.35	2,000,000	4	750,000	1,013,657	1.35
Big Plant	0.40	500,000	5			
	0.25	- 600,000	6			



12.11.6 Example of Risks and Assumptions

In water supply projects, assumptions could include:

i) Timely availability of land for construction of water intake;
ii) Timely disbursement of funds;
iii) Stable political situation;
iv) Timely completion of the dam; and
v) Regular adjustment of water tariffs.

In terms of risks, these assumptions would be formulated as follows:

i) Land not timely available for construction;
ii) Funds not timely disbursed;
iii) Political instability;
iv) Dam not ready in time;
v) Water tariffs not regularly adjusted.

12.11.7 Sustainability and Pricing

Economic viability of a water supply project (WSP) depends on its financial viability, i.e., sustainability of the project's financial returns. The economic analysis of projects should include an analysis of the financial viability of project agencies and environmental sustainability of project inputs and outputs.

There are other dimensions of sustainability are institutional sustainability and technical sustainability. With regard to institutional sustainability, the financial impact of the project on the concerned institutions needs to be evaluated and the question to be asked is whether or not these institutions are able to pay the financial subsidies that may be needed for the WSP to survive. Economic analysis may also suggest institutional changes or policy measures needed to sustain the financial and economic benefits generated by the project. Technical sustainability is looked after as part of the analysis of alternatives and determination of the least-cost option, which is done in the early project preparation or feasibility stage.

12.11.8 Distribution Analysis and Impact on Poverty

Water supply provision, especially in the rural areas and shantytowns in urban areas, is considered to be important for poverty reduction. The poverty-reducing impact of a project is determined by evaluating the expected distribution of net economic benefits to different groups such as consumers and suppliers, including labour and the government.

Table 12-6: Sample Output from Financial Analysis

OKOLLO TC WATER SUPPLY SCHEME - OPERATION AND MAINTENANCE COSTS

	Item Description	Units	Base Year (2010)	Intermediate Year (2020)	Ultimate Year (2030)
1.0	**Water sales**				
1.1	Proportion of Population served	No.	50%	60%	80%
1.2	Population served	No.	9,626	17,153	33,915
1.3	Water demand	m^3/day	193	343	678
1.4	Estimated Water Sales/month	m^3/month	5,775	10,292	20,349
	Total Water Sales per month	**UGX/month**	**4,732,723**	**18,208,559**	**77,724,386**
2.0	**Energy Costs**				
2.1	**General**				
2.1.1	Pumping rate	m^3/hr	10.00	10.00	10.00
2.1.2	Daily Pump Hours run	hrs	19	34	68
2.1.3	Daily Energy Consumption (12KW)	KWh/day	231	412	814
2.2	**Diesel**				
2.2.1	Generator Fuel Consumption	L/KWH	0.30	0.30	0.30
2.2.2	Daily fuel consumption	L/day	69.30	123.50	244.19
2.2.3	Diesel Price	UGX/L	2,100.00	3,200.00	4,300.00
2.2.4	Total Energy costs (Diesel)	UGX/month	4,366,127	11,856,430	31,500,438
2.3	**Grid Power**				
2.3.1	Cost of energy	UGX/KWH	450	550	650
2.3.2	Total Energy costs (Grid)	UGX/Month	-	-	-
2.4	**Solar Power**				
2.4.1	Cost of energy	UGX/KWH	-	-	-
2.4.2	Total Energy Costs (Solar)	UGX/Month	-	-	-
	Total Energy Costs (Diesel)	**UGX/month**	**4,366,127**	**11,856,430**	**31,500,438**
	Total Energy Costs (Grid)	**UGX/month**	**-**	**-**	**-**
	Total Energy Costs (Solar)	**UGX/month**	**-**	**-**	**-**

	Item Description	Units	Base Year (2010)	Intermediate Year (2020)	Ultimate Year (2030)
3.0	**Asset Maintenance Costs**				
3.1	Production Wells	UGX/year	220,000	474,963	1,025,411
3.2	Pumping Stations (Pumps, Generators, CPs, Wiring)				
3.2.1	Diesel	UGX/year	4,000,000	8,635,700	18,643,829
3.2.2	Solar	UGX/year	3,800,000	8,203,915	17,711,637
3.2.3	Grid	UGX/year	-	-	-
3.3	Pipework (Transmission and Distribution)	UGX/year	5,424,500	11,711,089	25,283,362
3.4	Storage Tanks	UGX/year	3,030,000	6,541,543	14,122,700
3.5	Public Kiosks	UGX/year	240,000	518,142	1,118,630
	Total Asset Maintenance Costs (Diesel)	**UGX/month**	**1,076,208**	**2,323,453**	**5,016,161**
	Total Asset Maintenance Costs (Solar)	**UGX/month**	**1,059,542**	**2,287,471**	**4,938,478**
	Total Asset Maintenance Costs (Grid)	**UGX/month**	**-**	**-**	**-**
4.0	**Depreciation (economic life reciprocal)**				
4.1	Production Wells	UGX/year	550,000	1,187,409	2,563,526
4.2	Pumping Stations (Pumps, Generators, CPs, Wiring)				
4.2.1	Diesel	UGX/year	1,000,000	2,158,925	4,660,957
4.2.2	Solar	UGX/year	11,000,000	23,748,175	51,270,529
4.2.3	Grid	UGX/year	-	-	-
4.3	Pipework (Transmission and Distribution)	UGX/year	18,081,667	39,036,962	84,277,873
4.4	Storage Tanks	UGX/year	10,100,000	21,805,142	47,075,667
	Total Asset Depreciation Costs (Diesel)	**UGX/month**	**2,477,639**	**5,349,037**	**11,548,169**
	Total Asset Depreciation Costs (Solar)	**UGX/month**	**3,310,972**	**7,148,141**	**15,432,300**
	Total Asset Depreciation Costs(Grid)	**UGX/month**	**-**	**-**	**-**

	Item Description	Units	Base Year (2010)	Intermediate Year (2020)	Ultimate Year (2030)
5.0	**Chemical Costs**				
5.1	Chlorination	UGX/Month	1,836,545	3,272,869	6,471,020
6.0	**Operator Fixed Costs**	**UGX/month**	**1,600,000**	**3,454,280**	**7,457,531**
7.0	**Management Style**				
7.1	Proposed Split (Operator/Board)	No.	0.95	0.95	0.95
7.2	Management Fees(Operator)	UGX/month	4,496,087	17,298,131	73,838,167
7.3	Water Board Retained Funds	UGX/month	236,636	910,428	3,886,219
8.0	**Operator Surplus (Mgt Fees - Costs)**				
8.1	Surplus (Diesel)	UGX/Month	(4,382,793)	(3,608,901)	23,393,017
8.2	Surplus (Solar)	UGX/Month	(0)	8,283,511	54,971,137
8.3	Surplus (Grid)	UGX/Month	-	-	-
9.0	**Diesel**				
9.1	Management Fees (Operator)	UGX/Month	(4,163,654)	(3,428,456)	22,223,366
9.2	Water Board (Investment)	UGX/Month	(219,140)	(180,445)	1,169,651
10.0	**Solar**				
10.1	Management Fees (Operator)	UGX/Month	(0)	7,869,336	52,222,580
10.2	Water Board (Investment)	UGX/Month	(0)	414,176	2,748,557
11.0	**Grid**				
11.1	Management Fees (Operator)	UGX/Month	-	-	-
11.2	Water Board (Investment)	UGX/Month	-	-	-
12.0	**Community Financing**				
12.1	Proposed Tariff	UGX/jerry can	20	44	95

13	Assumptions		
13.1	Nominal Interest Rate [%]	8%	
13.2	Transport for fuel/litre [UGX]	100	
13.3	Price of diesel [UGX]	2,200	
13.4	USD Rate	1,950	
13.5	Depreciation	10	
13.6	Construction Costs [UGX]		
13.7	Additional virtual investment costs to meet demand deficit from 2020 at the 2010 value [UGX]		
13.8	Future (2020) value of virtual investment [UGX]		
13.9	Investment cost per capita in initial year [UGX]		
14	Cost of stationary [UGX/month]	100,000	
14.1	Water board allowances	30,000	
14.2	Scheme manager / accountant salary [UGX/month]	300,000	
14.3	Plumber / meter reader salary + Transport [UGX/month]	250,000	
14.4	Number of standpipes	36	
14.5	Kiosk attendand fee/ 25L Jerican	10	
14.6	Proposed tarrif / 25 L Jerican	50	

15		Infrastructure cost	Annual maitenance costs [% of cost]
15.1	Intake works, earthworks, water Storage (concrete, Steel Tanks, treatment works), piping, buildings and fences, and access roads		1%
15.2	Pumps, control panels and generators, water meters, chemical dosing gear, instruments and testing apparatus		5%
15.3	Water kiosks, latrines		2%
		-	
16	Extensions	2010	2020
16.1	Number of new connections	-	11
16.2	Cost per connection		-
16.3	cost of new connections	-	-
16.4	Cost of connections for the year	-	-
17.0	Chemical Costs		
17.1	Chlorine	UGX/m3	318
17.2	Alum	UGX/m3	200

12. 11 Project Financing Methods

12. 11.1 Introduction

Common financing methods are: capital budgeting, loans, shares and bonds.

12. 11.2 Capital Budgeting [Capital Rationing]

When multiple proposals have passed the ranking process, the firm must decide which projects can be accepted with the available funds in its capital budget. Capital rationing is the process involved with the selection of projects where there are limits to the available capital for investments in a given period.

12. 11.3 NPV Approach to Capital Rationing

NPV is frequently used as a criterion to rank projects competing for the available funds under conditions of capital rationing. Project with the highest NPV are accepted in descending order until the available funds are used up.Consider the project below:

Table 12-7 Example on NPV Approach to Capital Rationing

	Cost [$]	NPV [$]	Rank
Project D	250,000	30,000	1
Project E	300,000	25,000	2
Project B	150,000	20,000	3
Project C	200,000	15,000	4
Project A	100,000	10,000	5
TOTAL	**1,000,000**	**100,000**	

If the capital budget of the firm is equal to the amounts shown below, the following combinations of projects would be accepted based on the NPV criterion. All things being equal, the firm will select all projects that can be included in the budget, provided the firm's objective is met: in this case to maximise the total NPV of the projects in the proposed capital budget.

Solution

Budget [$]	Projects	Total NPV [$]
1,000,000	D,E,B,C,A	100,000
900,000	D,E,B,C	90,000
800,000	D,E,B,A	85,000
600,000	D,B,C	65,000
400,000	D,B	50,000
300,000	D [or A & C]	30,000

12. 11.4 Profitability Index Approach

When the investment costs of projects differ significantly, the profitability index [PI] is the best ranking criterion, because it measures each project's return per dollar invested. Using this approach under capital rationing the objective is to select those projects which in combination have the highest weighted profitability index. Consider again the five projects from the proceeding example, ranked by their profitability index.

Table 12-8 Example on Profitability Index Approach

	Cost [$]	PI	Rank
Project B	150,000	1.133	1
Project D	250,000	1.120	2
Project A	100,000	1.100	3
Project E	300,000	1.083	4
Project C	200,000	1.075	5

Assuming again that the capital budget of the firm is limited to one of the amounts shown below, the acceptable combinations of projects, using the PI criterion, are as follows:

Solution

Budget [$]	Projects	Weighted PI
1,000,000	B, D, A, E, C	1.100
900,000	B, D, A, E	1.106
800,000	B, D, A, E	1.106
600,000	B, D, C	1.108
400,000	D, B	1.125
300,000	A, B [or D]	1.120

The weighted PI of a group of projects is computed by dividing the sum of the present values of the projects in that group by the sum of their investment costs. The group of projects having the highest weighted PI that can be purchased with a given capital budget is selected by the firm, all other things being equal. Notice that the optimal combination of projects leaves a capital surplus when the capital budget is either $900,000 or $300,000.

12. 11.5 IRR Approach to Capital Rationing

All projects whose rate of return, IRR, is greater or equal to the firm's minimum acceptable rate of return [MARR], subject to the available funds.

12. 11.5 External Capital Rationing

Periodically, business firms finance their capital requirements by obtaining funds externally through loans, stocks and bonds. These will tend to demand a higher return from the firm because of the perceived higher risk associated with larger amounts of capital investment.

12. 11.6 Linear Programming for Capital Rationing

Linear programming is a widely used technique to maximize or minimize a linear function subject to linear constraints. The general form of a linear programming model consists of an objective function and a number of equations or constraints as follows:

Maximize $$P = C_1x_1 + C_2x_2 + \ldots + C_nx_n$$ **Equation 12-16**

Subject to:

$$a_{11}x_1 + a_{12}x_2 + \ldots + C_{1n}x_n \leq B_1$$

$$a_{21}x_1 + a_{22}x_2 + \ldots + C_{2n}x_n \leq B_2$$

$$a_{m1}x_1 + a_{m2}x_2 + \ldots + C_{mn}x_n \leq B_m$$

$$x_1 \geq 0,\ x_2 \geq 0\ \ldots,\ x_n \geq 0$$

When used in capital budgeting, the objective function is specified to maximize the desired objective of the firm, such as the net present value. The constraints [Bs] may include the available capital, space, time, and other limited resources. The Xs are the decision variables, i.e., projects whose values are determined when the linear programming model is solved. The constants in the objective function, Cs, are the NPVs of each project under consideration. The other constraints are the coefficients that specify the limits of available resources with respect to each project. When the model is solved, the optimum solution indicates which projects, Xs, must be included in the budget to maximize the desired objective. You can use graphical methods or Ms Excel solver to solve these linear equations.

12.12 Compounding, Discounting and Economic Equivalence

12.12.1 Definitions

Economics is the social science concerned with the production, consumption and distribution of goods and services [Webster, 2003]. The study of economic can take two perspectives: macroeconomics and microeconomics.

Macroeconomics is concerned with the economy as a whole: national income, employments, flow of money, investments, etc.

Microeconomics on the other hand is concerned with the study of small segments of the economy such as, firms [supply] and individual consumers [demand]. Firms are interested in maximizing profit or net revenue [revenue-cost] while individuals are interested in maximizing utility [net benefit].

Engineering economics employs economic theory, mathematical programming, and statistical analysis to formulate and solve problems concerning with the evaluation and selection of capital projects at enterprise level.

12.12.2 Time Value of Money

Because money can earn interest during the time it is invested, a future return is worth less at the present time. Conversely, an amount of dollars invested now will be worth more when the principal and its accumulated interest are received n years from now. This relationship between interest rate and the value of money over time is the basis of two fundamental concepts in engineering economy: compounding and discounting.

12.12.3 Simple Interest

Interest is assessed only on initial principal. This concept is not frequently used in project evaluation.

$$F = P(1+i_n) \quad \textbf{Equation 12-17}$$

Where:
F = future value of the investment
P = present value of the investment
i = annual interest rate
n = number of interest periods per year.

12.13 Compound Interest

12.13.1 Introduction

The relationship between the future value of the investment and the present value of investment can be given by:

$$F = P(1+i)^n \quad \textbf{Equation 12-18}$$

12.13.2 Nominal and Effective Interest Rates

Because the formula for compounding is usually based on an interest period of one year, this annual interest rate of compounding is called the *nominal* rate of interest. When the compounding period is less than a year, the actual or *effective* interest is greater than the nominal rate due to the more frequent compounding. The following formula is used to calculate the effective rate of interest (i_e):

$$i_e = \left[1 + \frac{i}{m}\right]^m - 1$$ **Equation 12-19**

Where: i = Nominal interest rate per year
m = Number of interest periods per year.

Period Interest rate

$$i_p = \left[\frac{i}{m}\right]$$ **Equation 12-20**

The period over which the compounding takes place must match the period for which rate is calculated.

12.13.3 Quarterly Compounding

If the annual interest rate is say 10% and compounding occurs quarterly, the effective annual interest rate is:

$$i_e = \left[1 + \frac{0.1}{4}\right]^4 - 1 = 10.38\%$$ **Equation 12-21**

12.13.4 Monthly Compounding

If the annual interest rate is say 10% and compounding occurs monthly, the effective annual interest rate is:

$$i_e = \left[1 + \frac{0.1}{12}\right]^{12} - 1 = 10.47\%$$ **Equation 12-22**

12.13.5 Daily Compounding

If the annual interest rate is say 10% and compounding occurs daily, the effective annual interest rate is:

$$i_e = \left[1 + \frac{0.1}{365}\right]^{365} - 1 = 10.52\%$$ **Equation 12-23**

12.13.6 Continuous Compounding

When interest is compounded continuously, i.e. an infinite number of times per year, the annual rate of continuous compounding is:

$$i_e = e^r - 1$$ **Equation 12-24**

Where e = base of natural logarithm [2.7182…]
r = annual interest rate

12.13.7 Discounting

If the future value of an investment [or payment] is known, we can derive its present value, given an interest rate and the number of compounding periods by solving for P.

$$P = \frac{F}{(1+i)^n}$$ **Equation 12-25**

The process used in converting the future value to its present value is called discounting.

12.13.8 Interest Rate Formulas

The following assumptions made in engineering economy studies: Capital expenditure occurs at the beginning of the year and costs are paid or income is received at the end of the year.

The following symbols and their definitions will be applied:

P = Present value of an amount of money
i = annual interest rate
n = number of annual interest periods
F = Future value at the end of the periods
A = Annual amount, in a series of equal payments or receipts, made at the end of each year.

12.13.9 Single Payment Compound Amount Factor

When an amount of money P is invested at a given interest rate i for n years, the future value of the investment is given by:

$$F = P(1+i)^n$$ **Equation 12-26**

Graphically, it can be seen that the present value is an outlay and it is represented by a downward arrow at time zero, 0, while, its future value is an inflow represented by an upward arrow.

The amount in brackets, which is used to multiply P in order to derive F, is called the *compound interest factor for single payment* [F/P] It is usually designated [F/P, i, n].

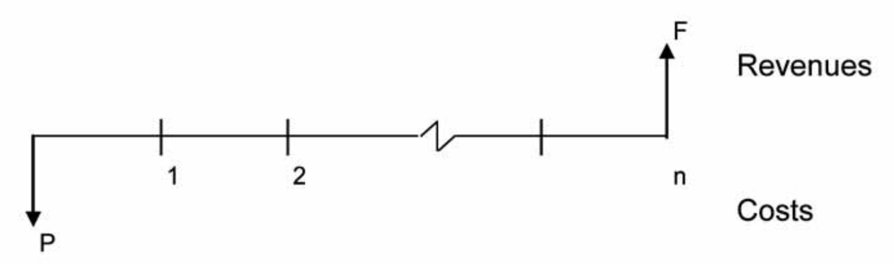


Figure 12-1 Single Payment and Future Value

12.13.10 Single Payment Present Worth Factor

When the future value of the investment F is known, the amount of money P to be invested at a given interest rate i for n years is given by:

$$P = \frac{F}{(1+i)^n}$$ **Equation 12-27**

The amount in brackets, which is used to multiply F in order to derive P, is called the *present worth factor for single payment* [P/F] It is usually designated [P/F, i, n].

12.13.11 Uniform Series Present Worth Factor

It is often required to know the present value of a series of equal payments [or receipts] over a number of years at a given rate of interest. Such a series is known as an annuity.

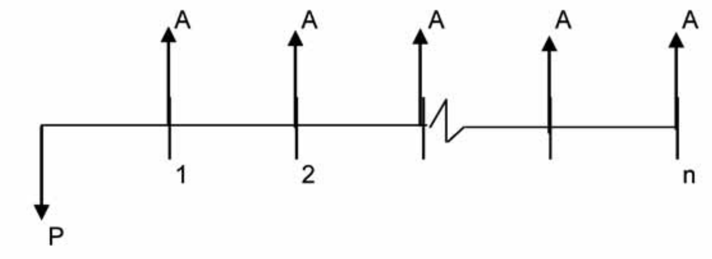


Figure 12-2 Present Value of Uniform Series

$$P = A\left[\frac{(1+i)^n - 1}{i(1+i)^n}\right] \qquad \textbf{Equation 12-28}$$

The amount in brackets is called the *present worth factor for uniform series* [P/A] It is usually designated [P/A, i, n].

12.13.12 Uniform Series Capital Recovery Factor

This is used to find the annual receipts [annuity] that will be generated in the future over the life of the investment at a given rate of interest.

$$A = P\left[\frac{i(1+i)^n}{(1+i)^n - 1}\right] \qquad \textbf{Equation 12-29}$$

The amount in brackets is called the *capital recovery factor* [A/P] It is usually designated [A/P, i, n].

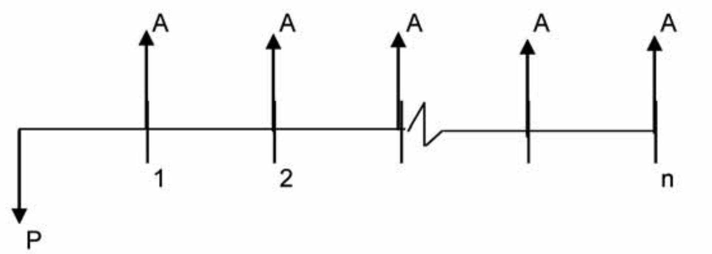


Figure 12-3 Uniform Series of Preset Value

12.13.13 Uniform Series Compound Amount Factor

The amount of future capital investment achieved by setting aside each year a certain amount of savings which over time accumulate to the required amount.

$$F = A\left[\frac{(1+i)^n - 1}{i}\right] \qquad \textbf{Equation 12-30}$$

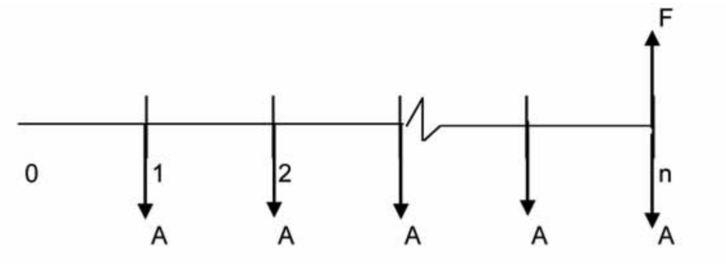


Figure 12-4 Uniform Series of Future Value

The amount in brackets is called the *uniform series compound amount factor* [F/A] It is usually designated [F/A, i, n].

12.13.14 Uniform Series Sinking Fund Factor

When the future capital expenditures for a certain activity or project are known, the required amount to be invested annually at a given interest rate must be determined.

$$A = F\left[\frac{i}{(1+i)^n - 1}\right] \quad \textbf{Equation 12-31}$$

The amount in brackets is called the *sinking fund factor for uniform series* [A/F] It is usually designated [A/F, i, n].

12.13.15 Uniform Gradient Series

When an investment generates a uniform series of either increasing or decreasing payments by a constant amount, rather than discount each payment separately, the equivalent payment of the series is computed and then discounted using the uniform series compound factor.

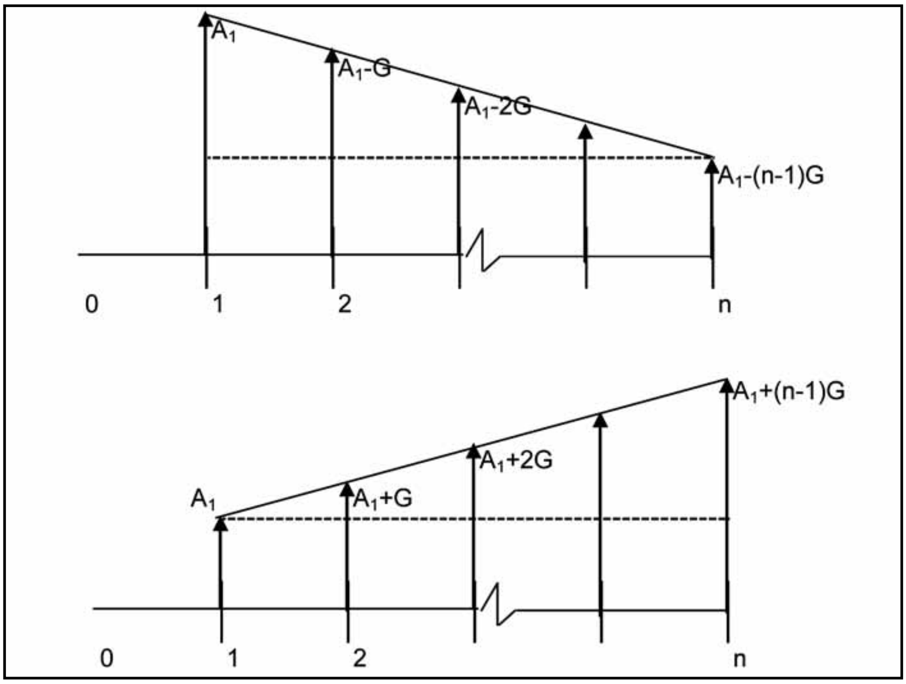


Figure 12-5 Uniform Gradient Series

The equivalent cash flows is obtained from the equation

$$A = A_1 \pm G\left[\frac{1}{i} - \frac{n}{(1+i)^n - 1}\right]$$ **Equation 12-32**

Where G = the constant amount of change or gradient per period.

The amount in brackets is called the *uniform gradient series factor* [A/G] It is usually designated [A/G, i, n].

12.13.16 Geometric Gradient Series

The annual payments of certain investments may increase [or decrease] by a constant percentage rate over the time horizon of the investment as shown in Figure 12-6. In this case, the present value of a geometric gradient series is obtained by the following formula:

$$P = \frac{A_1}{1+g} \pm [P/A, i^*, n]$$ **Equation 12-33**

Where g = annual rate of growth [or decline] of payment
i* = adjusted interest rate

$$i^* = \frac{1+i}{1+g} - 1$$ **Equation 12-34**

The amount in brackets is called the *geometric gradient series factor [P/A]*.

$$P/A = \frac{(1+i^*)^n - 1}{i^*(1+i^*)^n}$$ **Equation 12-35**

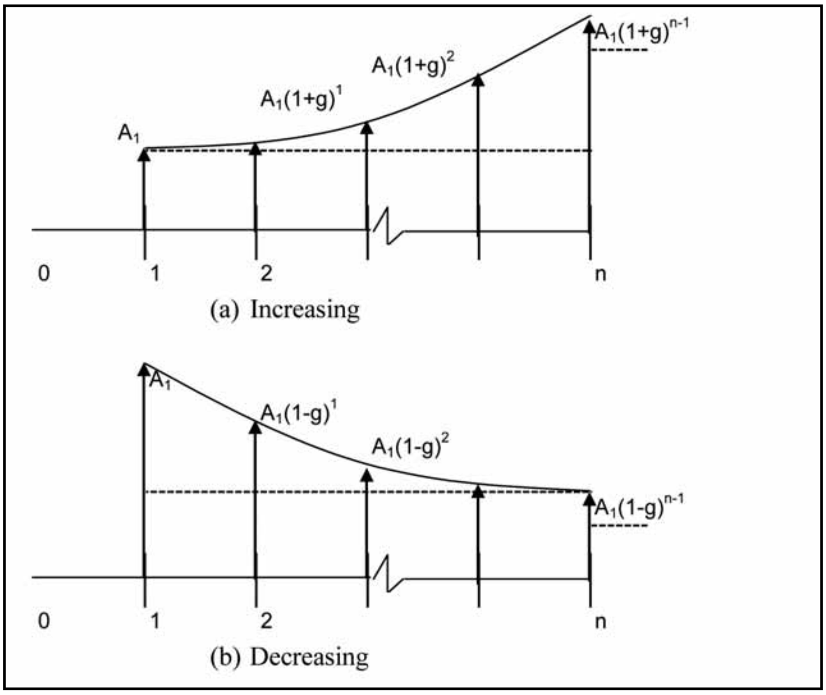


Figure 12-6 Geometric Gradient Series

12.13.17 Economic Equivalence

Given an interest rate, economic equivalence involves the conversion of amounts of money occurring at different time periods to an equivalent amount.

Example

A person inherits $10,000 and invests it in a bank account that guarantees to pay 9% interest for 3 years. Calculate the future value of this investment, if interest is compounded: (a) annually; (b) quarterly; (c) daily; (d) continuously.

Solution:

a) Annually

$$F = P(1+i)^n \text{; P = \$10,000, i = 9\%, n = 3; } F = 10000(1+0.09)^3 = \$12,950$$

(b) Quarterly

$$i_e = \left[1+\frac{0.09}{4}\right]^4 - 1 = 9.31\% \text{; P = \$10,000, i = 9.31\%, n = 3; } F = 10000(1+0.0931)^3$$

$$= \$13,061$$

(c) Daily

$$i_e = \left[1+\frac{0.09}{365}\right]^{365} - 1 = 9.42\% \text{; P = \$10,000, i = 9.416\%, n = 3; } F = 10000(1+0.09416)^3$$

$$= \$13,099$$

(d) Continuously

$$i_e = e^r - 1 \text{; } i_e = 2.718^{0.09} - 1 = 9.417\%\text{; } F = 10000(1+0.09417)^3 = \$13,099$$

[Tables are frequently used instead of these formulae]

12.14 Cash Flow Analysis and Inflation

12.14.1 Introduction

The justification of a capital project requires a comparison of the cost outlays and the cash inflows over its useful life. Cash flow analysis involves the determination of the magnitude and timing of the annual cash flow of a project based on estimated costs and forecasted sales of the product or service being generated.

12.14.2 Economic Modeling

Engineering economy studies focus on the selection of the best among two or more alternatives. Although there are many approaches used to determine the best investment, engineering economic analysis always involves the solution of an equation or model where all but one of the variables is known. An example of an economic model in engineering economy is the annual payment present worth factor formula

$$P = A[P/A, i, n] \qquad \textbf{Equation 12-36}$$

Where, given A, i, and n, the present worth, P, of an investment is computed. The challenge facing the decision maker, therefore, is the estimation of the independent variables of annual returns [A], interest rate [i], and the life of the investment, however must be forecasted based on statistical data or other information [Cassimatis, 1992].

12.14.3 Estimating the Life of an Investment

The useful life of an investment may be estimated from:

a) historical data or experience of the company for similar investments
b) technological forecasting [e.g., obsolescence of current products or processes], and
c) Market research by forecasting changes in demand for products and services in the future. An expected decline in demand for product X will shorten the useful life of an investment in this product.

12.14.4 The Interest Rate

The rate of interest, i, is established by the company's finance department. It represents the "cost of capital" of the firm. It is usually defined as the minimum annual rate of return [MARR] that a company must earn on an investment.

12.14.5 Cash Flow

Cash flow is the figure obtained after all annual costs, taxes, [but before depreciation] have been deducted from sales revenues.

Table 12-9 Cash Flow Analysis of a Project

Item	Amount
Sales Revenue	100,000
Less Costs	80,000
Earnings	20,000
Less depreciation	8,000
Earning before taxes	12,000
Taxes [40%]	4,800
Earnings after tax	7,200
Cash flow	**15,200**

This can be expressed as:

$$CF = E[1-T] + DT$$ **Equation 12-37**

Where E = earnings [sales – costs]
D = depreciation
T = income tax rate.

12.14.6 Inflation and Its Measurement

When estimating cash flows for a project, the analyst must take into account the impact of inflation on the sales and operating costs which the cash flows are derived. Inflation is a term used to describe the rising level of prices in the economy over time. Inflation impacts sales, costs, and interest rates. There are many indexes for inflation, but the most commonly used in economic analyses are the implicit deflator of the Gross National Product [GNP], the Producer Price Index [PPI] and the Consumer Price Index [CPI].

CPI is designed to measure only changes in prices for a fixed market basket of goods and services purchased by selected representative groups of consumers. This figure is compiled and is available on national and regional basis from the National Bureau of Statistics. It is reported monthly and annually.

12.14.7 Inflation and its Impact of Cash Flows

When discounting cash flows, the interest rate is usually stated in nominal terms. Therefore cash flows should be estimated in future current dollars, reflecting the rising trend in prices and costs embodied in the forecasted vales of sales and operating costs. The alternative is to use constant dollars.

It is important to take anticipated inflation into account when estimating the expected cash flows of an investment. The appropriate index of inflation should be used to adjust future sales and costs from which the cash flows are derived. In general, the CPI is the best index for sales and cost data that occur at retail level [Cassimatis, 1992]. The PPI measures inflation of costs and sales at the primary market level of activity. The GNP implicit price deflator is the best index from an overall measure of inflation at the national level.

When the cash flows are in constant dollars, consistency requires that the interest rate be in real terms. Nominal interest can be converted into real terms as follows:

$$i_R = \frac{1+i}{1+p} - 1 = \frac{i-p}{1+p}$$ **Equation 12-38**

Where i_R = interest rate in real terms
i = nominal interest rate
p = annual rate of inflation

When p is small i_R is approximately equal to i-p. In practice, it is neither convenient nor advantageous to use constant dollars in cash flow analysis, because the present value of the cash flow in both cases will be the same.

Example

In Table 12-9 12-9, it was assumed that the expected sales and costs of a project would be \$100,000 and \$80,000, respectively, per year, without considering inflation. Suppose that the expected annual rate of inflation will be 5% per year in the next 5 years, and the inflation rate will be the same for both sales and costs.

Table 12-10 Cash Flow Analysis in Current Dollars [Inflation Rate: 5% p.a.].

Year	1	2	3	4	5	
F/P, 5%, n	1.0500	1.1025	1.1576	1.2155	1.2763	
Sales Revenue - Costs	21,000	22,050	23,153	24,310	25,526	
Less depreciation	8,000	8,000	8,000	8,000	8,000	
Earning before taxes	13,000	14,050	15,153	16,310	17,526	
Taxes [40%]	5,200	5,620	6,061	6,524	7,010	
Earnings after tax	7,800	8,430	9,092	9,786	10,515	
Cash flow	15,800	16,430	17,092	17,786	18,515	
P/F, 10%,n	0.9091	0.8264	0.7513	0.6830	0.6209	
DCF	14,364	13,579	12,841	12,148	11,497	64,428

Table 12-11 Cash Flow Analysis in Constant Dollars [Interest Rate = 10%; Inflation Rate: 5% p.a.].

Year	1	2	3	4	5	
Current Dollars	15,800	16,430	17,092	17,786	18,515	
Constant Dollars	15,048	14,902	14,764	14,633	14,507	
P/F,4.762%,n	0.9545	0.9112	0.8697	0.8302	0.7925	
DCF	14,364	13,578	12,841	12,148	11,497	64,428

12.15 Depreciation and Taxation

12.15.1 Introduction

Capital projects usually include such assets as machinery, equipment, and buildings, which have relatively long lives. When the asset is purchased, its costs treated as an outlay, but under current tax laws, the firm is permitted to deduct a depreciation allowance from the income it generates each year during its depreciable life, thereby increasing it's after tax income.

A capital asset, such as a plant or a piece of equipment, losses its initial value over time due to:

a) physical deterioration [wear and tear];
b) functional depreciation,
c) technology obsolescence [technological depreciation], or
d) Depletion.

12.15.2 Definitions

Depreciation [or depreciation expense] refers to the periodic allocation of the cost of a tangible long-term asset over its useful life.

Obsolescence: is the process of becoming out of date.

Residue value [salvage value or disposal value]: the estimated net scrap, salvage, or trade-in value of a tangible asset at the estimated date of its disposal.

Depreciable cost: the cost of an asset less residual value

12.16 Depreciation Methods

12.16.1 Introduction

The traditional methods of depreciation are:

a) straight-line;
b) units-of-production method;
c) declining balance method;
d) sum-of-the-years-digits;
e) sinking fund method or; and
f) a variation of these.

12.16.2 Straight Line Depreciation

When the straight-line method is used to allocate depreciation, the depreciable cost of the asset is spread evenly over the life of the asset. The annual amount of depreciation is computed by dividing the initial cost of the asset by the number of years of its estimated life:

$$D = \frac{C_O}{N}$$ **Equation 12-39**

Where C_o is the cost of the asset and N is its life in years.
If salvage value S is expected then the annual amount of depreciation is computed as follows:

$$D = \frac{C_O - S}{N}$$ **Equation 12-40**

Under this method, the annual rate of depreciation is constant.

12.16.3 Units-of-Production Method

This method is based on the assumption that depreciation is solely the result of use and that the passage of time plays no role in the depreciation process.

12.16.4 Declining Balance Method of Depreciation

This method is also called reducing balance method, diminishing value method. This method is designed to allow larger amounts of depreciation in the earlier years and smaller amounts in the later years of the asset's life. Given a rate of depreciation d expressed in percentage terms, the depreciation in year t is obtained as:

$$D_t = dB_{t-1} \qquad \textbf{Equation 12-41}$$

Where B = is the book value of the asset.

When the rate of depreciation used is double the rate under straight line, the method is known as the *double-declining-balance* method of depreciation. In this case, the constant depreciation rate is:

$$d = \frac{2}{N} \qquad \textbf{Equation 12-42}$$

And the amount of depreciation in any year is obtained from

$$D_t = \frac{2}{N} B_{t-1} \qquad \textbf{Equation 12-43}$$

Another approach to accelerated depreciation is the *double-declining balance switching to straight-line* method. The switch is justified when the straight-line depreciation in a certain year is greater that the declining-balance method.

$$D_t = dB_{t+1} < \frac{B_{t-1} - S}{N - (t-1)} \qquad \textbf{Equation 12-44}$$

A rule of thumb used in determining the year to switch is based on the formula:

$$\frac{N}{2} + 1 \qquad \textbf{Equation 12-45}$$

For a 5-year asset, the switch occurs in year 4, i.e., (5/2) +1 = 3.5 rounded to 4. For a 10 year asset, the year to switch is year 6: (10/2) +1 = 6.

12.16.5 Sum-of-the-Years-Digits Method

The rate of depreciation in this method progressively decreases over the life of the investment.

$$d_t = \frac{N - (t-1)}{N(N+1)/2} \qquad \textbf{Equation 12-46}$$

Where d_t is the annual rate of depreciation in year t. For example, a 3-year asset's depreciation rates would be computed as follows. Sum-of-the-year's digits: 1+2+3 = 6. Annual depreciation rate: 3/6 for the first year, 2/6 for the second year, 1/6 for the third year. The amount of annual depreciation in year t is given by:

$$D = d_t (C_o - S) \qquad \textbf{Equation 12-47}$$

This method of depreciation is said to approximate more closely the actual decline in the value of certain assets over time than straight-line depreciation.

12.16.6 Effect of Different Depreciation Methods on Cash Flow

The method that yields the greatest present value of cash flows for a given period should be used. Accelerated depreciation tends to reduce taxes in the early years, thereby increasing the cash flows in those years. For this reason firms, generally, prefer using the accelerated method of depreciation.

12.16.7 Example

Supposing a delivery truck cost $10,000 and has an estimated residual value of $1,000 at the end of its estimated useful life of five years. What is the depreciation each year of this asset using the:

a) Straight line depreciation.
b) Assume that the truck used 20,000 km for the 1st year, 30,000 km for the second year, 10,000 for the third, 20,000 km for the fourth, and 10,000 for the 5h year. Calculate the annual depreciation using the units-of production method.
c) use sum-of-digit method
d) Use declining balance method

Solution

a) Straight Line Method

Year	Cost	Depreciation	Cum Depreciation	Book Value
0	10,000		-	10,000
1	-	1,800	1,800	8,200
2	-	1,800	3,600	6,400
3	-	1,800	5,400	4,600
4	-	1,800	7,200	2,800
5	-	1,800	9,000	1,000
Salvage	1,000	9,000		

b) Units – of - Production

Year	Cost	Km	Depreciation	Cum Depreciation	Book Value
0	10,000			-	10,000
1	-	20,000	2,000	2,000	8,000
2	-	30,000	3,000	5,000	5,000
3	-	10,000	1,000	6,000	4,000
4	-	20,000	2,000	8,000	2,000
5	-	10,000	1,000	9,000	1,000
Salvage	1,000	90,000	9,000		

c) Sum–of–digits Method

Year	Cost	Rate	Depreciation	Cum Depreciation	Book Value
0	10,000			-	10,000
1	-	0.333	3,000	3,000	7,000
2	-	0.267	2,400	5,400	4,600
3	-	0.200	1,800	7,200	2,800
4	-	0.133	1,200	8,400	1,600
5	-	0.067	600	9,000	1,000
Salvage	1,000	1.000	9,000		
15					

c) Declining Balance Method using 1.5 x straight line rate

Year	Cost	Rate	Depreciation	Cum Depreciation	Book Value
0	10,000			-	10,000
1	-	0.300	3,000	3,000	7,000
2	-	0.300	2,100	5,100	4,900
3	-	0.300	1,470	6,570	3,430
4	-	0.300	1,029	7,599	2,401
5	-	0.300	1,401	9,000	1,000
Salvage	1,000	1.500	9,000		

Chapter 13

DESIGN REPORTS

13.1 General

The design of a water supply scheme involves preparation and presentation of among others design reports. The reports serve the following main functions:

i) To record and present all information concerning a project in a clear and concise manner for future reference, and for the information of planners and others parties interested in the project.
ii) To present the basic data, information, design criteria, assumptions and conclusions regarding the project, to enable planners and other parties interested in the project, to make appropriate decisions and supervise the implementation of the project.

13.2 Reporting Stages

13.2.1 Introduction

The form and content of a design report will largely, depend on the nature and complexity and size of the project. Large and complex projects require reporting during the design stages:

i) Inception Report
ii) Feasibility Study Report
iii) Preliminary Design Report
iv) Detailed Design Report
v) Tender documents
vi) Engineer's Estimate

For small and uncomplicated schemes, one detailed design report may be sufficient. The contents of each report are presented separately in this section, and should be adjusted to suite the requirements for a particular project.

13.2.2 Inception Report

The Inception report presents the following:

i) the mobilization and establishment status of the Consultant;
ii) the specific staffing plan;
iii) the updated work plan the Consultant proposes to follow in carrying out the assignment, based on the Consultants initial findings;
iv) Review of Methodology;
v) details of any constraints or inputs required from the employer; and
vi) such remarks as are deemed appropriate including the works done so far.

The Consultant shall carry out a comprehensive literature review immediately after signing the contract and will present his/her findings in the inception report. Literature review shall cover scholarly articles, books and other sources (e.g. dissertations, conference proceedings, policy statements, manuals, procedures, strategy papers, and existing laws) relevant to design/construction and related areas of research, or theory. The Consultant shall then provide a description, summary, and critical evaluation of each work. The purpose of the literature review is to offer an overview of significant literature published on the topic/project.

13.2.3 Feasibility Study Report

A feasibility Study (economic and conceptual design) is a requirement after the project identification process. There are usually a number of alternative technical solutions to be considered from which the most economically viable alternative is determined. During the feasibility study it is established whether for the best alternative solution chosen:-

i) the project conforms with the country's development objectives and priorities
ii) the relevant policy framework is compatible with the achievement of the projects' objectives;
iii) the project is technically sound and that it is the best of the available technical alternatives;
iv) the project is administratively workable;
v) there is adequate demand for the projects output;
vi) the project is economically justified and financially viable;
vii) the project is compatible with the customs and traditions of the beneficiaries;
viii) the project is environmentally (ecologically and socially) sound; and that
ix) the project is compatible with urban/regional master plans.

This feasibility study report should include:

i) General arrangement and layout of proposed project;
ii) Performance characteristics of the proposed project elements and components;
iii) Preliminary cost estimates based on life cycle approach;
iv) Plan for phased implementation;
v) Implementation schedule;
vi) Environmental, socio-economic impact of the proposed development; and
vii) Recommendation of the best alternatives for detailed design.

13.2.4 Preliminary Design Report

The consultant shall prepare and submit a report on preliminary design to the client for approval, which shall contain the project design brief and outline design drawings.

The report shall also include:

i) A geotechnical report;
ii) An Environmental and Social Impact Assessment Report; and
iii) Topographic And Cadastral Survey Report.

A geotechnical report may be required. This report shall present:

i) the soil conditions and characteristics of the site;
ii) Field test results: the geology of the area and field investigations: drilling bore holes, standard penetration tests, undisturbed samples, determination of ground water results;
iii) Laboratory tests: sieve analysis, Atterberg limits, moisture content, shear strength, specific gravity, consolidation, chlorides, sulphates, PH values, CBR, etc;

An Environmental and Social Impact Assessment Report will be required. The report shall summarize:

i) the effect of the project investment on the environment and recommend appropriate solutions to forestall any disagreeable effects resulting from the investment;
ii) the positive and negative impacts of the project;
iii) Environmental Management Plan to mitigate adverse environmental effects that that project may have;
iv) community's role in the project in so far as the project influences their lives, especially for women, children and the elderly, and quantify the benefits which would accrue to them during

and after the construction of the project; and

v) Approval from the National Environmental Management Authority.

Topographic And Cadastral Survey Report may be required. This report will:

i) confirm the boundaries of the plot;
ii) size of the plot;
iii) existing ground profiles;
iv) position of existing buildings and features and the location of existing services; and
v) Where adjustments to the boundaries are found necessary, the Consultant shall recommend the appropriate action.

13.2.5 Detailed Design Report

This report shall include detailed Engineering Report on all investigations, technical, economic, environmental and social investigations, structural design and calculations.

The detailed designs would be expected to include:

i) Statistical analysis of data collected for the population and demand projections, hydrological data and hydrogeological, meteorological data etc;
ii) Least cost lay-outs for different components of the project, i.e. treatment plants, hydraulic and structural works;
iii) Structural and stability computations of different structures;
iv) Calculations for pumps, motors, power generators and other machinery and equipment;
v) Engineering analysis for deciding the most economic size of delivery mains; and
vi) Hydraulic computations for the distribution system.

Detailed Drawings would include:

i) Index plan showing overall layout of project;
ii) Schematic diagram showing levels of salient components of the project (may not necessarily be to scale) ;
iii) Detailed plans and sections in scale for the headwords, treatment plants, clear water storage tank, pumping station, etc., in a scale 1 :20 to 1: 100 depending on the details and size of work;
iv) Detailed structural plans for structures, intake treatment plant, clear water reservoir etc., in a scale of 1: 20;
v) Index plan of the. distribution system normally in an appropriate scale;
vi) Longitudinal sections of the delivery main and details of appurtenances in scales;
a) Horizontal scale 1: 500 to 1: 5000 depending on distance and details
b) Vertical scale 1:20 to 1:100 depending on the terrain surface undulations.

13.2.6 Tender Documents

Tender documents form the basis on which a civil engineering Contractor will prepare his tender and carry out and complete the contract works. These documents should detail all the requirements of the project in a comprehensive and unambiguous way. These documents also identify all the rights and duties of the main parties to the contract. Collectively they constitute a binding contract, where by the contractor undertakes to construct works in accordance with the details supplied by the Engineer and the Employer agrees to pay the Contractor in stages during the execution of the works in the manner prescribed in the contract.

The tender documents should be based on the standard PPDA documents Vol 1 – 4 and FIDIC for contracts and, will include:

i) Volume 1 (Special conditions of contract and General conditions of contract);
ii) Volume 2 (Bill of quantities);
iii) Volume 3 (Specifications); and
iv) Volume 4 (Drawings).

The tender documents may be based on other standard bidding documents as may be advised by the Client.

13.2.7 Engineer's Estimates

The Consultant will be expected to have prepared a confidential pre-bid cost estimate and submitted the same to Client together with the bidding documents. As far as possible this should be based on unit costs derived from recent projects of a similar magnitude, complexity and remoteness from or proximity to ports or major urban areas.

Detailed estimates of recurrent costs should be based on unit costs provided by the operating authority or from schemes of a similar size and nature. Anticipated revenue should be based on the recommendations made regarding tariff structures or provided by the operating authority

13.3 Contents of the Inception Report

Abbreviations and Acronyms
List of Tables
List of Figures
Executive Summary

1 Background
1.1 Introduction
1.2 Objectives of the Consultancy Services
1.3 Scope of Services
1.4 Description of Project Area –location, climate, topography, geology
1.5 Consultant's Mobilisation
1.5.1 Team Composition
1.5.2 Support/Extra Staff
1.5.3 Facilities
1.6 Reconnaissance Visit Report
1.7 Outline of the Inception Report

2 Review of the Methodology
2.1 Introduction
2.2 Activity Overview
2.3 Mobilisation and Reconnaissance
2.4 Inception Report
2.5 Data Collection and analysis
2.6 Feasibility Study Report
2.7 Detailed Design Report

3 Literature Review

4 Work Program
4.1 Introduction
4.2 Expected Outputs/Deliverables
4.3 Completion and Submission of Reports and other Deliverables

5 Appendices
5.1 Appendix 1: Terms of Reference
5.2 Appendix 2: Table of Contents of Feasibility Study Design Report
5.3 Appendix 3: Table of Contents of Detailed Design Report
5.4 Appendix 4: Household questionnaire
5.5 Appendix 5: Community Leaders' Questionnaire
5.0 Appendix 6: Minutes of Client-Consultant Meeting

13.4 Contents of the Feasibility Study Report

The feasibility study report shall include abbreviations and acronyms, list of tables, list of, figures, and executive summary

CHAPTER 1 INTRODUCTION

In this chapter the history of the proposed project and the explanation on how the project fits in the regional and national annual and long term water supply programme should be given. Previous studies overview i.e. master plans, pre-feasibility studies etc. should be given. The objectives of the consultancy services, the scope of the consulting services is also presented.

CHAPTER 2 PROPOSED PROJECT

In this chapter the project for implementation is selected. The previous proposed plans should be confirmed or revisions presented. The chapter should cover:

- The planning period
- Project objectives
- Areas to be covered, i.e. consumers to be covered
- Population Projections
- Demand projections
- Ranking of the selected project
- Recommended development plan
- Existing infrastructure: administration, education institutions, health institutions, commerce and industry, transport, telephone, electricity, etc.
- Existing water supplies/sources: location, source, and ownership, consumers, technical and economic assessment of the supplies, reliability and constraints

CHAPTER DESIGN CRITERIA

The design criterion is stated in this chapter. This will cover:

3.1 Design Horizon
3.2 Domestic and Non-Domestic Water Demand Rates
3.3 Design Economic Life
a. Water supply

i) Intake works and raw water transmission mains
ii) Treatment works
iii) Transmission and distribution
iv) Pumping stations and Mains
v) Water storage

3.5 Sanitation

3.6 Financial analysis
i) Comparison of Project Alternatives
ii) Investment Cost Estimates
iii) Operation and Maintenance Cost Estimates
iv) Cash flow of water supply scheme

CHAPTER 4 WATER SUPPLY SCHEME COMPONENTS

The water supply scheme component alternative are identified and evaluated. Recommended supply scheme component are selected for design. This chapter should cover:

a. WATER DEMAND
   i. Peak day demand
   ii. Loss Factors
   iii. Projected Demand [domestic, institutional, commercial, industrial, other, total demand]

b. WATER SOURCES
   i. Ground water sources
   ii. Surface water sources
   iii. Recommended source

c. SANITATION

   i. Proposed interventions
   ii. Sustainability aspects for sanitation

d. ENVIRONMENTAL IMPACT ASSESSMENT

   i. Screening
   ii. Scoping
   iii. Environmental impact study
   iv. Environmental impact assessment

e. SOCIAL ECONOMIC STUDY
   i. Economy and income situation
   ii. Willingness and ability to pay for water
   iii. Conclusions

f. COST ESTIMATES
   i. Construction costs
   ii. Operation and Maintenance Costs
   iii. Miscellaneous costs
   iv. Unit Cost

CHAPTER 5 INSTITUTIONAL FRAMEWORK

In this chapter various organizations that might be involved in the detailed design, construction and the operations and maintenance of the project need to be defined. Their roles and responsibilities should be clearly spelt out. This should cover:-

a. Organization and management
b. Staffing implications, and training requirements
c. Future staffing and training plans
d. Financial history of the present organization responsible for running the system
e. Tariffs and charging system for water supply systems
f. Tentative financing plan, i.e. source of funds etc.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

In this chapter the feasibility of the project is stated and recommended actions to be taken made. This should cover:

a. Summary of the study and the proof that the project is feasible, economically, technically, financially, socially, culturally, environmentally and institutionally.
b. Recommended action for the successful implementation and operation of the project.

13.5 Contents of the Preliminary Design Report

Abbreviations and Acronyms
List of Tables
List of Figures
EXECUTIVE SUMMARY

There should be a Project Summary sheet, giving information about:

- Geographic location
- Water source
- Water treatment
- Water demand
- Distribution system
- Implementation stages
- Annual costs.

Further, an O&M Expenditure Summary sheet should be attached, giving all the estimated costs for O & M for the initial year and for the following 20 years in stages of 5 years, plus the revenue expected, all the above being on a present cost basis.

A map to scale 1:20,000 or more detailed if possible, should show the scheme boundary.

The Summary should give in concise way information about the background of the project, with respect to:

Demography, technical, economic and geographical data of the recommended scheme and of any existing supply. It should also describe the alternative solutions studied and the reasons for discarding the alternative schemes and selecting the one in question.

1 CHAPTER 1 INTRODUCTION

This chapter should include:

1. Background;
2. Previous schemes, reports or studies;
3. Scope of the present report;
4. Description of the scheme are: geographic location, climatic conditions, geological and topographical conditions; and
5. Reasons of the study.

2 CHAPTER 2 EXISTING WATER SUPPLY SCHEMES

The chapter on existing water schemes should discuss:

a. Ownership, Location, Source
b. Purpose of the schemes
c. Type and number of consumers

d. Level of Service
e. Existing constraints and reliability of the supply, both at present and in future
f. Technical and economical assessment of the supply system.

3 CHAPTER 3 SOCIO-ECONOMIC STUDY

The socio-economic chapter should contain information on:

a. Commerce and industry
b. Transport
c. Agriculture/forestry
d. Health facilities
e. Education
f. Administration
g. Economic and income situation
h. Willingness to pay

4 CHAPTER 4 WATER DEMAND

This is a very important chapter and must address the following:

a. Design period and Design horizon
b. Population
c. Livestock
d. Institutions
e. Industry
f. Commerce
g. Houses and gardens
h. Other Specific Water demands
i. Total water demand.

5 CHAPTER 5WATER SOURCES

As relevant, the alternative water sources available should be discussed under the following headings:

1. Groundwater
2. Surface water
3. Springs
4. Other sources
5. Recommended source
6. Estimate of the total capacity of all available sources within an economic
7. Estimate of the total population, which can be served by the available water sources within the remainder of this century.

6 CHAPTER 6 DESIGN CRITERIA

This is regarded as an important chapter and must contain the following:

a. Review of pertinent design criteria and proposals contained in this in water design manual
b. Identify and comment on those criteria not being followed clearly indicating the reasons for this
c. Summarize both the design criteria and the proposals being adopted
d. All parameters and assumptions that will be used in the design computation
e. All data to be used, methods of analysis, etc.

This chapter should have a detailed description of:

- Design Horizon
- Domestic and Non-Domestic Water Demand Rates

- Design Economic Life
- Water supply
- Intake works and raw water transmission mains
- Treatment works
- Transmission and distribution
- Pumping stations and Mains
- Water storage

□ Financial analysis

- Comparison of Project Alternatives
- Investment Cost Estimates
- Operation and Maintenance Cost Estimates

□ Sanitation

- The unlined pit latrine
- The lined pit latrine
- The ECOSAN toilet
- Septic tank

7 CHAPTER 7 WATER INTAKE STRUCTURES

The structures proposed foe water intake should be described with respect to:

a. Location
b. Intake (river, pond, lake, etc.)
c. Dam Well Spring
d. Raw water mains

The above information should be accompanied by general sketches or drawings and Preliminary Computations

8 CHAPTER 8 PUMP STATIONS

Where pumping is required, this must be clearly described depending upon type:

a. Raw water pump station
b. Clear water pump station
c. Rising mains

The above are to be accompanied by general sketches or drawings and by respective preliminary computations.

9 CHAPTER 9 TREATMENT PLANT

This chapter should include:

- Location of the treatment plant
- Water quality of the source
- Demanded water quality for supply
- Recommended treatment processes
- Reasons for deviation from WHO standards, (if any)
- Flow plan for the proposed treatment processes.

10 CHAPTER 10 TRANSMISSION AND DISTRIBUTION

This chapter should cover:

10.1 Gravity Mains
- 10.1.1 Raw water mains
- 10.1.2 Clear water mains.

10.2 Distribution System proposals should include:
- 10.2.1 Recommended distribution system including preliminary computations and system sketches.
- 10.2.2 Alternative systems considered and reasons for not applying them
- 10.2.3 Choice of materials for pipes, fittings, etc., and reasons for excluding any of them
- 10.2.4 Proposed implementation schedule
- 10.2.5 Implementation time schedule
- 10.2.6 Phasing of execution proposed.

11 Chapter 11 ENVIRONMENTAL IMPACT ASSESSMENT

An environmental impact assessment report covering:

- 11.1 Screening
- 11.2 Scoping
- 11.3 Environmental impact study
- 11.4 Environmental impact assessment

12 CHAPTER 12 COST ESTIMATES

These should be to the detail required and include:

- Costs for design, construction and supervision
- Costs for O & M
- Costs for management and administration
- Revenue expected,
- Financial Internal Rate of Return and Economical Rate of Return
- Conclusion regarding prices of the water and about the economy of the project, and
- Cash flow projections.

In addition a statement is required on Life Cycle Analysis and Costing, indicating the current state of information available and to what extent this is being taken into consideration.

13.6 Contents of the Detailed Design Report

Abbreviations and Acronyms
List of Tables
List of Figures

EXECUTIVE SUMMARY

There should be a Project Summary sheet, giving information about:
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- Water demand
- Distribution system
- Implementation stages
- Annual costs.

Further, an O&M Expenditure Summary sheet should be attached, giving all the estimated costs for O & M for the initial year and for the following 20 years in stages of 5 years, plus the revenue expected, all the above being on a present cost basis.

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c. Type and number of consumers
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e. Existing constraints and reliability of the supply, both at present and in future
f. Technical and economical assessment of the supply system.

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b. Transport
c. Agriculture/forestry
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f. Administration.
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4.5 Industry

4.6 Commerce
4.7 Houses and gardens
4.8 Others
4.9 Specific Water demands
4.10 Total water demand.

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As relevant, the alternative water sources available should be discussed under the following headings:

5.1 Groundwater
5.2 Surface water
5.3 Springs Other sources
5.4 Recommended source
5.5 Estimate of the total capacity of all available sources within an economic, and
5.6 Estimate of the total population, which can be served by the available water sources within the remainder of this century.

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This is regarded as an important chapter and must contain the following:

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e. All data to be used, methods of analysis, etc.

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- Design Economic Life
- Water supply
  - Intake works and raw water transmission mains
  - Treatment works
  - Transmission and distribution
  - Pumping stations and Mains
  - Water storage
- Financial analysis
  - Comparison of Project Alternatives
  - Investment Cost Estimates
  - Operation and Maintenance Cost Estimates
- Sanitation
  - The unlined pit latrine
  - The lined pit latrine
  - The ECOSAN toilet
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7.3 Dam
7.4 Well
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7.6 Raw water mains

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8.2 Clear water pump station
8.3 Rising mains

The above are to be accompanied by general sketches or drawings and by respective preliminary computations.

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This chapter should include:

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9.2 Water quality of the source
9.3 Demanded water quality for supply
9.4 Recommended treatment processes
9.5 Reasons for deviation from WHO standards, (if any), and
9.6 Flow plan for the proposed treatment processes.

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10.1 Raw water mains
10.2 Clear water mains.
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- 10.3.1 Recommended distribution system including preliminary computations and system sketches.
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- 10.3.3 Choice of materials for pipes, fittings, etc., and reasons for excluding any of them
- 10.3.4 Proposed implementation schedule
- 10.3.5 Implementation time schedule
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An environmental impact assessment report covering:

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11.2 Scoping

11.3 Environmental impact study
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These should be to the detail required and include:

12.1 Costs for design, construction and supervision
12.2 Costs for O & M Costs for management and administration
12.3 Revenue expected, Financial Internal Rate of Return and Economical Rate of Return
12.4 Conclusion regarding prices of the water and about the economy of the project, and
12.5 Cash flow projections.

In addition a statement is required on Life Cycle Analysis and Costing, indicating the current state of information available and to what extent this is being taken into consideration.

APPENDICES

Appendix 1: Hydraulic calculations

Appendix 2: Structural calculations

Appendix 3: Economic Calculations

13.7 Units

13.7.1 General

Only SI units are to be used in water supply reports and drawings in Uganda.

13.7.2 Basic SI Units

The most common SI units in water supply projects in Uganda are presented in Table 13-1.

Table 13-1: Basic SI Units of Measure

Measure	Unit	symbol
Length	metre	m
Mass	kilogramme	kg
Time	second	s

13.7.3 Derived and Supplementary SI Units

The derived and supplementary SI units most commonly used in water supply projects are as presented in Table 13-2.

Table 13-2: Derived and Supplementary SI Units

Measure	Unit	Symbol
Area	square metre	m^2
Volume	cubic metre	m^3
Density	kilogram per cubic metre	kg/m^3
Velocity	metres per second	m/s
Force	newton	N

Pressure/stress	Pascal	Pa
Power	Watt	W
Temperature	degree Celsius	°C
Angle	degree	°

13.7.4 Prefixes

Prefixes may be used as an alternative method for writing multiples and sub-multiples, as shown in Table 13-3.

Table 13-3: Prefixes

Figure	Multiple	Prefix	Symbol
One million	10^6	Mega	M
One thousand	10^3	kilo	k
One thousandth	10^{-3}	milli	m
One millionth	10^{-6}	micro	µ

13.8 SI Unit Symbols, Multiples, Sub-Multiples and Conversion Factors

Table 13-4 presents the commonly used units, their symbols, multiples and sub-multiples, together with the conversion factors between the SI system and the Imperial system.

Table 13-4: SI Unit Symbols, Multiples, Sub-Multiples and Conversion Factors

Quantity	Unit Symbol	Multiples And Sub-Multiples	Conversion Factors
Length Mass Time	m kg s	km, mm Mg, g, mg hour (h) day (d)	1m =3.281 ft = 39.370 inches 1kg = 2.205 lb = 35.274 oz 1h = 3.600 s
Temperature Angle Area	^{0}C ^{0} m^2	Minute (‘) Second (‘’) km^2, mm^2	°C = 0.556 (°F - 32) (1° = 60’) Note: 360° =1 circle 1 m^2 = 10.764 ft^2 1 acre = 4046.825 m^2
Volume Density Force	m^3 Kg/m^3 N	Mm^3, dm^3,L g/ml kN, MN	1 m^3= 35.315 ft^3 =219.969 galls UK 1L = 1dm^3 = 0.001 m^3 1 kg/m^3 = 0.062428 lb/ft^3 1 N = 0.010197 kp= 0.2248 lbf.
Pressure stress Velocity Power	Pa m/s w	MPa, kPa, MN/m, N/mm^2 km/h MW, kw	1Pa = 1 N/m^2 = 10.197 x 10^6 kPcm^2 = 10^{-5} bar = 0.145 x 10^{-3} lbf/in^2 (psi) 1 m/s = 3.281 ft/s = 2.237 miles/h 1N = 1 Nm/s = 1.341 x 10^3 hp (UK) = 3.412 Btu/h

Note that no plural “s” shall be added to any symbol e.g. write 10 km not 10 kms.

Table 13-5 Other Symbols

Symbol	SI Unit	Symbol	SI Unit
Kg	kilogramme	kPa	kilo Pascal
m^2	metre squared	kN	kilo Newton
Pa	Pascal	MPa	mega Pascal
m/s	metres per second	MN/m	Mega Newton per metre
w	watt	N/mm^2	Newton per millimetre squared
N	Newton	km	kilometre
mm	millimetre	Mg	megagram
g	gram	mg	milligram
oz	ounce	galls UK	gallons (UK)
lbf	pound force		

13.9 Formats for Reports and Drawings

13.9.1 Formats

The following format should be used for reports.

- A4 Size 210 mm x 297 mm

The following formats should be used for drawings:

- A1 Size 594 mm x 840 mm (mainly for construction drawings)
- A3 Size 297 mm x 420 mm (mainly for presentation purposes)

13.9.2 Scales

The following scales are recommended to be used for drawings.

- 1:2, 1.20, 1:200, 1:2000
- 1:5, 1.50, 1:500, 1:5000
- 1:10, 1.100, 1:1000, 1:10000

13.9.3 Symbols

Symbols recommended for use on drawings for water supply and related installations are shown in Appendix C

13.9.4 Pipeline Drawings

The plan and profile of a pipeline should be drawn on the same sheet, and to the same horizontal scale. The plan should be placed above the profile. The horizontal and vertical scales used should be 1:2,000 and 1:200 respectively, except where more detailed drawings are required. Pipeline chainages should start at the inlet end and run in the same direction as the flow of water.

A pipeline profile should show the ground levels of all surveyed points, and the pipeline invert levels at points where gradients change. Levels should be given in metres up to two places of decimal. Chainages should be given in whole metres only.The profile should also show the static headline, hydraulic gradient line in per cent, pipe materials, pipe sizes, pipe classes, design flows and ground materials. A pipeline plan should show enough topographic features to make it possible to find the surveyed lines at the time of construction. Distances from the pipeline to well-defined and prominent features should be given. The “North Point” should be indicated on all plans.

13.9.5 Standard Drawings

It is expected that in due course, the Directorate of Water Development, in its continuing efforts towards the goal of standardization, will develop, compile and publish documents containing the approved standard drawings (and related specifications), for the various water supply system components.

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APPENDICES

APPENDIX 1: POPULATION ESTIMATES

Appendix 1a: Population Estimates Growth Rates

Appendix 1b: Population Estimates For Some Sub Counties

Appendix1a: Population Estimates Growth Rates (Source UBOS)

Name of Region	Name of District	2012Estimated Population	Estimated Population Growth Rates		
			1969-80	1980-91	1991-02
NORTHERN	1. Abim	51,603			0.91
	2. Adjumani	375,900			5.04
	3. Amolatar	127,400			2.28
	4. Amuru	232,900			2.22
	5. Apac	581,100	3.17	3.38	2.71
	6. Arua	562,500	2.36	2.73	2.71
	7. Dokolo	183,200			2.83
	8. Gulu	396,400	1.81	2.05	2.29
	9. Kaabong	395,300			5.43
	10. Kitgum	419,000	2.42	1.33	3.21
	11. Koboko	236,800			4.92
	12. Kotido	233,300	4.08	1.76	5.27
	13. Lira	715,100	2.72	2.75	2.65
	14. Moroto	333,800	1.31	0.74	4.59
	15. Moyo	412,600	1.62	4.55	6.12
	16. Nyadri	413,300			
	17. Nakapiripirit	275,200			4.71
	18. Nebbi	565,900	1.27	2.79	2.12
	19. Oyam	378,900			2.79
	20. Pader	531,500			3.96
	21. Yumbe	545,300			6.29
	22. Arua Municipality	61,400			2.72
	23. Gulu Municipality	158,654			2.29
	24. Moroto Municipality	13,000			4.66
	25. Lira Municipality	112,157			2.65
CENTRAL	26. Kalangala	66,300	2.23	5.88	5.33
	27. Kampala	1,723,200	3.14	4.76	3.04
	28. Kayunga	358,800			1.62
	29. Kiboga	348,100	5.82	0.19	3.42
	30. Entebbe Municipality	83,200			3.4
	31. Luwero	440,400	2.59	0.79	2.1
	32. Lyantonde	80,200			1.61
	33. Masaka	775,300	3.1	2.71	0.81
	34. Masaka Municipality	74,700			0.85
	35. Nakaseke	193,500			2.72

Name of Region	Name of District	2012Estimated Population	Estimated Population Growth Rates		
			1969-80	1980-91	1991-02
	36. Nakasongola	156,500			1.72
	37. Mpigi	467,900	2.43	2.94	1.14
	38. Mukono	1,036,000	1.53	2.39	2.17
	39. Mubende	610,500	3.61	2.72	3.01
	40. Mityana	311,600			1.31
	41. Rakai	484,300	3.92	3.04	1.49
	42. Sembabule	219,700			1.63
	43. Wakiso	1,471,800			3.38
EASTERN	44. Amuria	503,200			6.67
	45. Budaka	201,200			2.22
	46. Bududa	185,000			3.16
	47. Bugiri	659,100			3.85
	48. Bukedea	186,400			3.43
	49. Bukwo	73,400			3.34
	50. Busia	297,700			2.31
	51. Butaleja	221,200			2.78
	52. Iganga	760,700	3.02	3.5	2.79
	53. Jinja	544,100	1.46	2.15	2.12
	54. Jinja Municipality	92,100			2.1
	55. Kaberamaido	199,200			3.4
	56. Kaliro	216,700			2.78
	57. Kamuli	766,000	2.19	2.98	2.68
	58. Kapchorwa	217,200	1.32	4.15	3.52
	59. Katakwi	176,800			3.26
	60. Kumi	413,300	2.19	0.11	3.56
	61. Manafwa	395,100			2.75
	62. Mayuge	461,300			2.87
	63. Mbale	441,300	2.68	1.41	2.32
	64. Mbale Municipality	94,300			2.31
	65. Namutumba	219,000			2.19
	66. Pallisa	544,000	2.46	2.86	2.86
	67. Sironko	365,000			2.1
	68. Soroti	546,700	2.18	-0.93	4.18
	69. Soroti Municipality	85,800			4.11
	70. Tororo	443,300	2.17	2.82	2.06
	71. Tororo Municipality	44,800			4.66

Name of Region	Name of District	2012Estimated Population	Estimated Population Growth Rates		
			1969-80	1980-91	1991-02
WESTERN	72. Buliisa	80,700			1.98
	73. Bundibugyo	346,000	3.32	0.35	4.06
	74. Bushenyi	894,100	2.36	3.08	1.64
	75. Fort Portal Municipality	47,800			1.25
	76. Hoima	549,000	2.24	3	3.82
	77. Ibanda	255,400			2.05
	78. Isingiro	420,300			2.32
	79. Kabale	498,100	1.25	2.17	0.67
	80. Kabale Municipality	73,000			0.68
	81. Kabarole	415,600	4.43	3.29	1.25
	82. Kamwenge	332,200			1.87
	83. Kanungu	252,200			1.7
	84. Kasese	748,000	5.06	1.94	2.92
	85. Kibaale	681,200	5.74	3.37	4.23
	86. Kiruhura	315,200			2.85
	87. Kisoro	254,300	0.95	3.53	1.16
	88. Kyenjojo	543,500			2.98
	89. Masindi	670,100	3.48	2.71	4.29
	90. Mbarara	445,800	4.07	2.75	1.71
	91. Mbarara Municipality	85,600			1.71
	92. Ntungamo	480,200			1.91
	93. Rukungiri	321,400	1.85	2.51	1.26
		35,277,814			

Appendix 1b: Population Estimates For Some Sub Counties (Source UBOS)

		2008	2009	2010	2011	2012
KAMPALA	**DISTRICT**	**1,480,200**	**1,533,600**	**1,597,800**	**1,659,500**	**1,723,200**
Central	Division	109,600	113,600	118,400	122,900	127,600
Kawempe	Division	326,400	338,100	352,300	365,900	379,900
Makindye	Division	377,300	391,100	407,300	423,100	439,300
Nakawa	Division	299,500	310,200	323,200	335,700	348,700
Rubaga	Division	367,400	380,600	396,600	411,900	427,700
		2008	**2009**	**2010**	**2011**	**2012**
KIBOGA	**DISTRICT**	**149,200**	**155,500**	**163,300**	**170,800**	**178,600**
Bukomero	Sub county	27,700	28,800	30,200	31,500	32,800
Butemba	Sub county	30,600	31,800	33,500	34,800	36,300
Dwaniro	Sub county	13,800	14,400	15,100	15,700	16,400
Kapeke	Sub county	14,200	14,800	15,500	16,200	16,800
Kibiga	Sub county	25,600	26,600	27,900	29,100	30,500
Kiboga	Town council	15,200	16,000	16,400	17,400	18,000
Kyankwanzi	Sub county	12,200	12,700	13,300	13,800	14,500
Lwamata	Sub county	26,400	27,500	28,700	30,000	31,300
Mulagi	Sub county	14,000	14,600	15,200	15,900	16,600
Muwanga	Sub county	16,300	16,900	17,700	18,500	19,300
Nsambya	Sub county	33,500	35,000	36,600	38,200	39,900
Ntwetwe	Sub county	31,600	32,900	34,400	35,900	37,400
Wattuba	Sub county	19,000	19,700	20,700	21,600	22,500
		2008	**2009**	**2010**	**2011**	**2012**
LUWERO	**DISTRICT**	**194,200**	**199,000**	**205,600**	**211,500**	**217,500**
Bamunanika	Sub county	28,900	29,500	30,400	31,200	32,100
Kalagala	Sub county	37,900	38,800	40,000	41,000	42,000
Kamira	Sub county	23,400	24,000	24,700	25,400	26,000
Kikyusa	Sub county	28,200	28,800	29,700	30,500	31,300
Zirobwe	Sub county	41,000	42,000	43,300	44,400	45,500
Bombo	Town council	19,400	19,900	20,500	21,000	21,600
Butuntumula	Sub county	33,900	34,700	35,800	36,700	37,600
Katikamu	Sub county	39,300	40,300	41,300	42,500	43,700
Luwero	Sub county	33,000	33,800	34,900	35,800	36,700
Luwero	Town council	27,300	28,000	28,800	29,500	30,300
Makulubita	Sub county	30,200	30,900	31,800	32,700	33,600
Nyimbwa	Sub county	32,000	32,700	33,700	34,600	35,500
Wobulenzi	Town council	21,900	22,500	23,100	23,700	24,300

		2008	2009	2010	2011	2012
MUBENDE	**DISTRICT**	**260,300**	**269,600**	**281,500**	**292,800**	**304,300**
Bagezza	Sub county	53,100	55,100	57,300	59,700	61,700
Butoloogo	Sub county	19,400	20,100	21,000	21,700	22,500
Kasambya	Sub county	90,700	94,000	97,900	101,400	105,500
Kitenga	Sub county	46,000	47,700	49,600	51,500	53,500
Kiyuni	Sub county	26,100	26,900	28,100	29,200	30,300
Madudu	Sub county	20,700	21,300	22,200	23,100	24,000
Mubende	Town council	19,900	20,600	21,400	22,200	23,100
Bukuya	Sub county	77,300	80,200	83,600	86,700	90,100
Kassanda	Sub county	73,000	75,600	78,700	81,800	84,800
Kiganda	Sub county	47,300	48,900	51,000	53,000	54,900
Myanzi	Sub county	51,800	53,600	55,800	58,000	60,200
		2008	**2009**	**2010**	**2011**	**2012**
MUKONO	**DISTRICT**	**459,400**	**471,000**	**486,800**	**501,000**	**515,500**
Buikwe	Sub county	32,600	33,500	34,500	35,600	36,500
Kawolo	Sub county	36,600	37,500	38,700	39,800	40,800
Lugazi	Town council	32,700	33,600	34,500	35,500	36,300
Najja	Sub county	36,100	37,000	38,000	39,100	40,300
Najjembe	Sub county	31,800	32,600	33,600	34,500	35,500
Ngogwe	Sub county	35,200	36,000	37,200	38,200	39,200
Njeru	Town council	59,900	61,300	63,200	64,900	66,800
Nkonkonjeru	Town council	13,000	13,400	13,700	14,000	14,400
Nyenga	Sub county	45,100	46,200	47,700	49,000	50,300
Ssi-Bukunja	Sub county	23,300	23,900	24,700	25,300	26,000
Wakisi	Sub county	39,000	40,000	41,300	42,300	43,500
Bugaya	Sub county	8,900	9,100	9,400	9,700	9,900
Busamuzi	Sub county	14,800	15,100	15,500	16,000	16,400
Bweema	Sub county	7,900	8,300	8,400	8,600	8,900
Nairambi	Sub county	18,000	18,400	19,100	19,600	20,100
Goma	Sub county	52,700	54,000	55,700	57,200	58,700
Kkome	Islands	11,300	11,600	11,900	12,300	12,700
Kyampisi	Sub county	33,500	34,300	35,300	36,200	37,300
Mukono	Town council	54,400	55,700	57,400	59,000	60,400
Nakisunga	Sub county	46,200	47,300	48,800	50,000	51,500
Nama	Sub county	38,500	39,500	40,700	41,900	43,000
Ntenjeru	Sub county	64,700	66,300	68,400	70,300	72,200
Kasawo	Sub county	36,100	37,000	38,200	39,200	40,300
Kimenyedde	Sub county	36,300	37,200	38,300	39,500	40,500

Nabaale	Sub county	33,200	34,000	35,000	35,900	36,900
Nagojje	Sub county	33,300	34,100	35,200	36,100	37,100
Ntunda	Sub county	15,600	16,000	16,500	17,000	17,500
Seeta-	Namuganga	38,500	39,400	40,700	41,800	42,900
		2008	**2009**	**2010**	**2011**	**2012**
NAKASONGOLA	**DISTRICT**	**72,000**	**73,500**	**75,600**	**77,400**	**79,300**
Kakooge	Sub county	23,100	23,600	24,100	24,700	25,200
Kalongo	Sub county	15,700	16,000	16,400	16,700	17,100
Kalungi	Sub county	19,400	19,700	20,200	20,700	21,100
Lwabyata	Sub county	12,100	12,300	12,600	12,900	13,200
Lwampanga	Sub county	24,200	24,700	25,500	25,700	26,400
Nabiswera	Sub county	16,300	16,600	17,000	17,400	17,700
Nakasongola	Town council	7,300	7,500	7,600	7,800	8,000
Nakitoma	Sub county	10,100	10,300	10,500	10,800	11,000
Wabinyonyi	Sub county	15,400	15,600	16,100	16,500	16,800
		2008	**2009**	**2010**	**2011**	**2012**
SSEMBABULE	**DISTRICT**	**99,600**	**101,400**	**104,100**	**106,500**	**108,800**
Lwemiyaga	Sub county	21,500	21,800	22,300	22,900	23,200
Ntusi	Sub county	13,400	13,800	14,000	14,300	14,700
Lugusulu	Sub county	24,200	24,500	25,100	25,600	26,100
Lwebitakuli	Sub county	54,200	55,100	56,500	57,600	58,800
Mateete	Sub county	58,000	59,200	60,600	61,800	63,000
Mijwala	Sub county	26,600	27,000	27,700	28,300	28,900
Sembabule	Town council	4,500	4,500	4,700	4,800	4,900
		2008	**2009**	**2010**	**2011**	**2012**
KAYUNGA	**DISTRICT**	**159,100**	**161,900**	**166,100**	**169,700**	**173,400**
Bbaale	Sub county	11,400	11,600	11,900	12,100	12,300
Galiraaya	Sub county	16,100	16,400	16,800	17,200	17,500
Kayonza	Sub county	50,300	51,200	52,400	53,500	54,400
Wabwoko-kitimbwa	Sub county	43,400	44,100	45,200	46,000	47,000
Busana	Sub county	54,000	54,800	56,300	57,500	58,700
Kangulumira	Sub county	49,100	50,000	51,100	52,100	53,200
Kayunga	Sub county	40,300	41,100	42,000	42,900	43,800
Kayunga	Town council	22,200	22,600	23,100	23,600	24,100
Nazigo	Sub county	44,000	44,800	45,800	46,700	47,700
		2008	**2009**	**2010**	**2011**	**2012**
MITYANA	**DISTRICT**	**144,700**	**146,700**	**150,000**	**152,600**	**155,300**
Butayunja	Sub county	11,500	11,600	11,800	12,000	12,200
Kakindu	Sub county	18,600	18,800	19,200	19,500	19,900

Maanyi	Sub county	33,200	33,700	34,400	35,000	35,500
Malangala	Sub county	21,000	21,200	21,700	22,100	22,400
Bulera	Sub county	56,200	56,800	57,900	58,800	59,800
Busimbi	Sub county	46,700	47,500	48,400	49,100	49,900
Kikandwa	Sub county	23,400	23,800	24,300	24,700	25,100
Mityana	Town council	37,400	38,000	38,700	39,300	39,900
Ssekanyonyi	Sub county	43,900	44,500	45,400	46,100	46,900
		2008	**2009**	**2010**	**2011**	**2012**
NAKASEKE	**DISTRICT**	**83,100**	**85,800**	**89,300**	**92,600**	**96,000**
Kapeeka	Sub county	29,400	30,300	31,400	32,600	33,500
Kasangombe	Sub county	21,400	22,100	22,900	23,700	24,600
Kikamulo	Sub county	26,800	27,600	28,700	29,700	30,700
Nakaseke	Sub county	23,900	24,700	25,700	26,500	27,400
Ngoma	Sub county	20,000	20,600	21,400	22,100	22,900
Semuto	Sub county	30,500	31,500	32,700	33,900	35,000
Wakyato	Sub county	14,800	15,300	15,800	16,300	17,000
		2008	**2009**	**2010**	**2011**	**2012**
KALANGALA	**DISTRICT**	**30,300**	**32,200**	**34,700**	**37,000**	**39,700**
Bujjumba	Sub county	9,600	10,200	11,000	11,700	12,500
Mugoye	Sub county	11,400	12,200	13,100	13,900	14,900
Bufumira	Sub county	12,100	12,900	13,900	14,800	15,800
Bufumira		2,700	2,900	3,100	3,400	3,600
Lulamba		9,400	10,000	10,800	11,400	12,200
Kyamuswa	Sub county	4,200	4,400	4,700	5,100	5,500
Mazinga	Sub county	5,000	5,300	5,700	6,000	6,500
		2008	**2009**	**2010**	**2011**	**2012**
MASAKA	**DISTRICT**	**394,900**	**397,900**	**404,500**	**409,400**	**413,800**
Bigasa	Sub county	39,400	39,800	40,300	40,700	41,100
Butenga	Sub county	47,900	48,000	48,800	49,300	49,800
Kibinge	Sub county	33,200	33,500	33,900	34,200	34,600
Kitanda	Sub county	27,300	27,600	27,900	28,200	28,500
Bukakata	Sub county	13,600	13,700	14,000	14,000	14,200
Buwunga	Sub county	40,600	40,900	41,500	41,900	42,300
Kabonera	Sub county	30,600	30,800	31,300	31,500	31,800
Kisekka	Sub county	46,700	46,900	47,700	48,200	48,600
Kkingo	Sub county	36,400	36,700	37,100	37,600	37,900
Kyanamukaaka	Sub county	47,900	48,200	48,900	49,400	49,800
Kyazanga	Sub county	44,200	44,600	45,200	45,700	46,200
Lwengo	Sub county	58,000	58,400	59,100	59,800	60,300

Malongo	Sub county	35,900	36,100	36,600	37,000	37,300
Mukungwe	Sub county	37,200	37,500	38,000	38,300	38,700
Ndagwe	Sub county	35,400	35,800	36,300	36,600	37,000
Bukulula	Sub county	43,100	43,500	44,000	44,500	44,900
Kalungu	Sub county	49,600	49,900	50,600	51,100	51,600
Kyamuliibwa	Sub county	32,800	33,000	33,500	33,800	34,200
Lukaya	Town council	15,000	15,100	15,300	15,500	15,600
Lwabenge	Sub county	29,700	29,900	30,400	30,700	30,900
Katwe/Butego	Division	18,200	18,400	18,700	18,900	19,000
Kimanya/ Kyabakuza	Division	22,200	22,400	22,600	22,900	23,100
Nyendo/senyange	Division	31,300	31,600	32,000	32,300	32,700
		2008	**2009**	**2010**	**2011**	**2012**
MPIGI	**DISTRICT**	**219,800**	**222,600**	**227,300**	**231,100**	**234,900**
Budde	Sub county	12,600	12,600	12,900	13,100	13,300
Bulo	Sub county	17,000	17,100	17,400	17,600	17,800
Kalamba	Sub county	21,100	21,400	21,700	22,100	22,400
Kibibi	Sub county	26,400	26,800	27,100	27,500	27,900
Ngando	Sub county	17,000	17,300	17,600	17,900	18,100
Kabulasoke	Sub county	46,000	46,500	47,400	48,000	48,700
Kyegonza	Sub county	39,800	40,200	40,900	41,500	42,000
Maddu	Sub county	27,200	27,600	28,000	28,400	28,800
Mpenja	Sub county	31,400	31,800	32,400	32,800	33,300
Buwama	Sub county	43,700	44,200	45,100	45,500	46,400
Kamengo	Sub county	32,700	33,100	33,700	34,200	34,700
Kiringente	Sub county	13,900	14,100	14,300	14,600	14,800
Kituntu	Sub county	21,700	21,900	22,300	22,600	22,900
Mpigi	Town council	37,300	37,600	38,300	38,900	39,400
Muduma	Sub county	22,800	23,200	23,500	23,900	24,200
Nkozi	Sub county	31,300	31,600	32,200	32,700	33,100
		2008	**2009**	**2010**	**2011**	**2012**
RAKAI	**DISTRICT**	**218,100**	**221,700**	**227,300**	**232,000**	**236,800**
Kakuuto	Sub county	29,500	29,900	30,700	31,400	31,700
Kasasa	Sub county	17,100	17,300	17,700	18,000	18,400
Kibanda	Sub county	17,200	17,600	17,900	18,200	18,500
Kifamba	Sub county	13,700	13,900	14,200	14,400	14,800
Kyebe	Sub county	17,800	18,100	18,500	18,800	19,200
Byakabanda	Sub county	15,300	15,600	15,800	16,300	16,600
Ddwaniro	Sub county	30,200	30,700	31,400	32,000	32,500
Kacheera	Sub county	19,700	20,100	20,500	20,900	21,200

Kagamba	Sub county	30,600	31,100	31,800	32,300	33,000
Kyalulangira	Sub county	30,900	31,300	32,100	32,600	33,300
Lwamaggwa	Sub county	36,900	37,500	38,300	39,100	39,700
Lwanda	Sub county	27,800	28,200	28,800	29,300	29,900
Rakai	Town council	6,600	6,700	6,900	7,000	7,200
Kabira	Sub county	29,000	29,400	30,100	30,700	31,200
Kalisizo	Sub county	31,000	31,500	32,100	32,600	33,300
Kasaali	Sub county	25,300	25,700	26,300	26,800	27,300
Kirumba	Sub county	26,300	26,800	27,400	27,900	28,400
Kyotera	Town council	8,400	8,600	8,800	9,000	9,300
Lwankoni	Sub county	15,600	15,900	16,200	16,500	16,700
Nabigasa	Sub county	20,700	20,900	21,400	21,800	22,200
		2008	**2009**	**2010**	**2011**	**2012**
WAKISO	**DISTRICT**	**554,300**	**576,400**	**604,200**	**630,700**	**658,200**
Kakiri	Sub county	38,000	39,600	41,400	43,200	45,100
Kasanje	Sub county	40,200	41,800	43,700	45,800	47,500
Katabi	Sub county	73,600	76,500	80,000	83,500	87,100
Masulita	Sub county	25,600	26,700	28,000	29,200	30,500
Namayumba	Sub county	33,300	34,600	36,200	37,800	39,300
Nsangi	Sub county	93,300	97,100	101,600	105,900	110,500
Ssisa	Sub county	58,200	60,500	63,400	66,000	68,900
Wakiso	Sub county	85,100	88,600	92,600	96,600	100,800
Wakiso	Town council	18,700	19,400	20,300	21,200	22,000
Division	A	40,800	42,500	44,500	46,300	48,400
Division	B	29,400	30,600	32,000	33,400	34,800
Busukuma	Sub county	34,700	36,100	37,800	39,400	41,100
Gombe	Sub county	50,900	52,900	55,300	57,700	60,200
Kira	Sub county	158,300	164,700	172,300	179,800	187,500
Nabweru	Sub county	133,200	138,600	145,100	151,300	157,800
Nangabo	Sub county	71,100	74,000	77,500	80,800	84,200
Ssabagabo-Makindye	Sub county	173,800	180,900	189,200	197,400	205,900
		2008	**2009**	**2010**	**2011**	**2012**
LYANTONDE	**DISTRICT**	**74,000**	**75,300**	**77,100**	**78,700**	**80,200**
Kaliiro	Sub county	16,800	17,200	17,600	18,000	18,200
Kasagama	Sub county	5,900	5,900	6,100	6,300	6,300
Kinuuka	Sub county	8,100	8,200	8,400	8,600	8,800
Lyantonde	Sub county	16,100	16,300	16,800	17,100	17,500
Lyantonde	Towncouncil	8,400	8,600	8,700	8,900	9,200
Mpumudde	Sub county	18,700	19,100	19,500	19,800	20,200

APPENDIX 2: WATER RESOURCES DATA

APPENDIX 2a: Surface Water Monitoring Network in Uganda

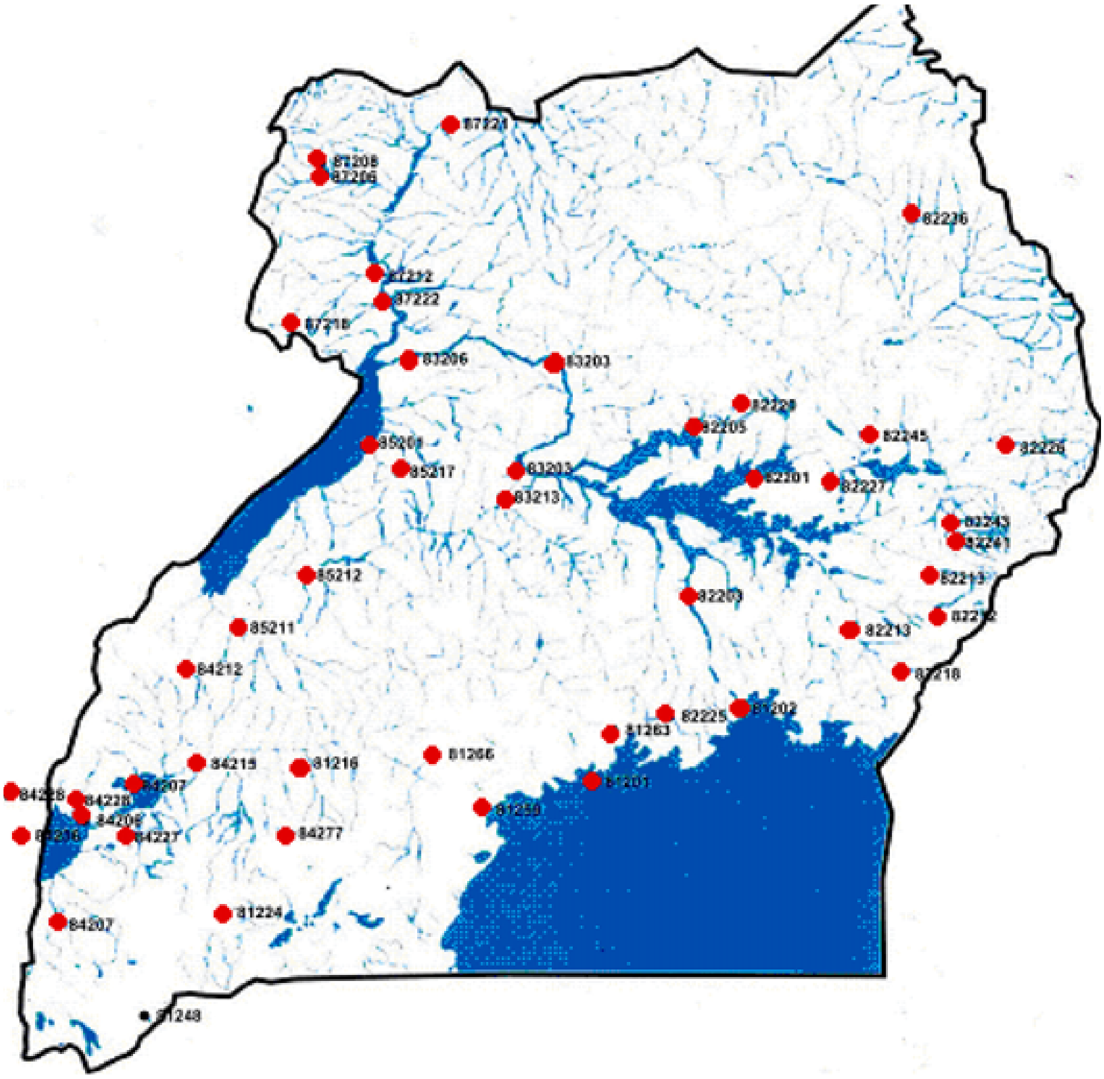
Appendix 2b: Groundwater Monitoring Network in Uganda

Appendix 2c: Water Quality Monitoring Network in Uganda

Appendix 2d: Rainfall Distribution in Uganda

Appendix 2e: Rain Gauge Locations in Uganda

APPENDIX 2a: Surface Water Monitoring Network in Uganda

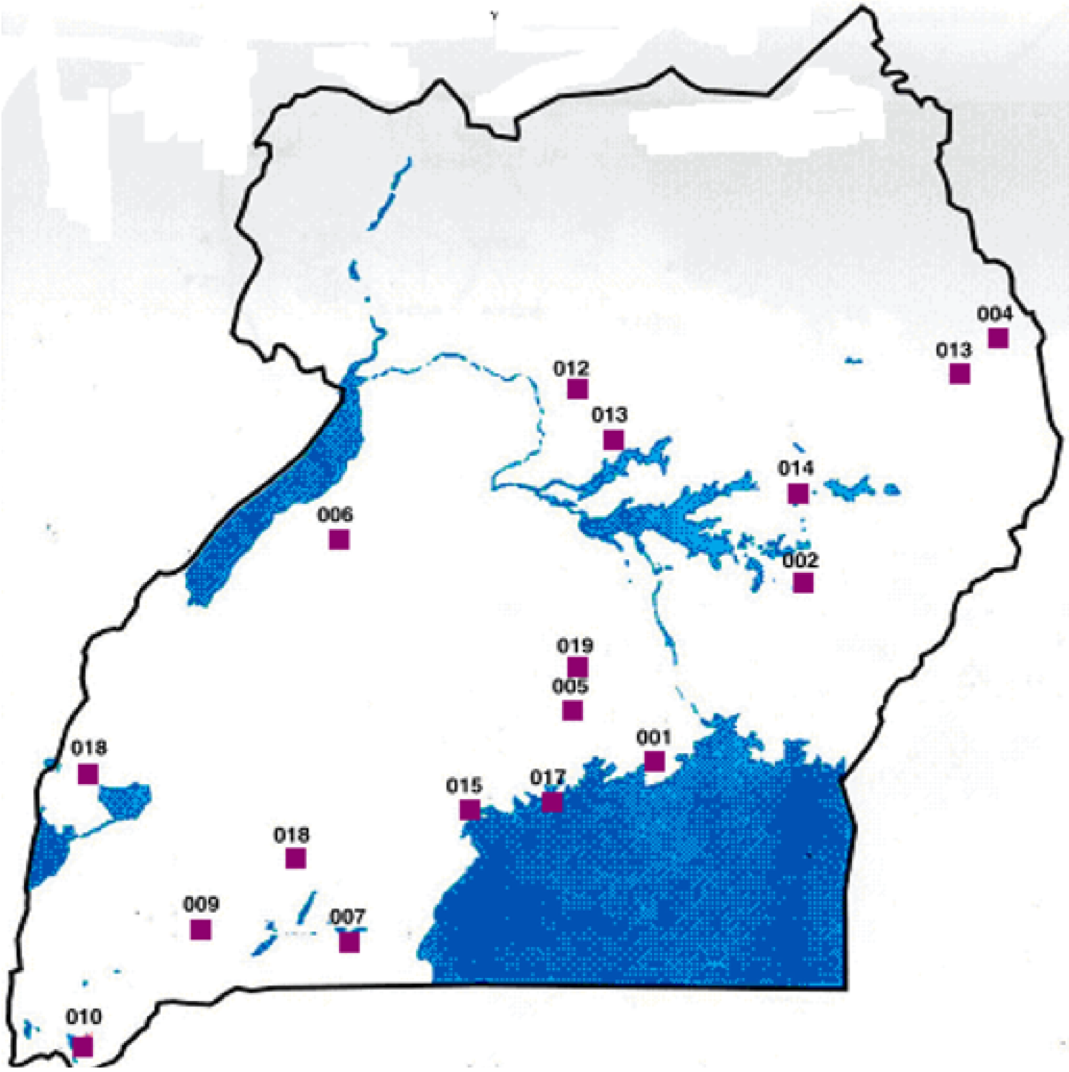


Source: UNWD Report, 2006

SURFACE WATER MONITORING STATIONS	
STATION ID	**STATION NAME**
82243	R. Sipi at Mbale Moroto Rd
82241	R. Simu at Mbale Moroto Rd
82240	R. Sironko at Mbale Moroto Rd
82231	R. Kelim (Greek) at Mbale Moroto Rd
82228	R. Namalu at Mbale Moroto Rd
82227	R. Kapiri at Kumi Soroti Rd
82222	R. Abuket at Kumi Serere Rd
82221	R. Agu at Kumi Serere Rd
82218	R. Malaba at Jinja - Tororo Rd
82217	R. Mpologoma at Budumba
82213	R. Namatala at Mbale Soroti Rd
82212	R. Manafwa at Mbale Tororo Rd
82203	R. Victoria Nile at Mbulamuti
81269	R. Sio at Luhalali near Bunadeti
84212	R. Mpanga at Kampala - Fort Portal Rd
83219	R. Kigwe at Semuto - Wobulenzi Rd
83218	R. Mayanja at Kapeeka - Kakunga Rd
82225	R. Sezibwa at Falls
81268	Nakivubo Channel - Railway Bridge
81267	Nakivubo Channel - 5th Street
81266	L. Wamala at Lubajja
81260	R. Kibimba at Kinoni - Mubende Rd
81259	R. Katonga at Kampala - Masaka Rd
81216	R. Kakinga Index Catchment
81202	L. Victoria at Jinja Pier
87218	R. Nyagak at Nyapea
87217	R. Albert Nile at Laropi
87212	R. Ora at Inde - Pakwach Rd
87208	R. Oru at Arua - Yumbe Rd
87206	R. Anyau at Arua - Moyo Rd
85217	R. Waki II at Biiso - Hoima Rd
85212	R. Nkussi at Kyenjojo - Hoima Rd
85211	R. Muzizi at Kyenjojo - Hoima Rd
83213	R. Kafu at Kampala - Gulu Rd
83212	R. Tochi II at Gulu - Atura Rd
83209	R. Kyoga Nile at Paraa
83203	R. Kyoga Nile at Masindi Port
85201	L. Albert at Butiaba

SURFACE WATER MONITORING STATIONS	
STATION ID	**STATION NAME**
82205	L. Kwania at Kachung
81201	L. Victoria at Entebbe Pier
81223	R. Kagera at Masangano
81224	R. Ruizi at Mbarara Water Works
81258	R. Bukora at Katera
81224	R. Ruizi at Mbarara Water Works
81258	R. Bukora at Katera
81248	R. Nyakizumba at Maziba
81270	R. Bukora at Mutukula - Kyotera
81271	R. Kisoma at Mutukula - Kyotera Rd
81272	R. Ruizi at New Waterworks
82201	L. Kyoga at Bugondo Pier
84206	L. Edward at Katwe
81271	R. Kisoma at Mutukula - Kyotera Rd
81274	R. Kisoma Upper Stream at Kyotera
82201	L. Kyoga at Bugondo Pier
82245	R. Akokorio at Soroti - Katakwi Rd
82252	R. Omunyal Upper at Tiririri Rd
82254	R. Mpologoma at Tirinyi-Mbale Rd
83206	R. Kyoga Nile at Kamdini
82220	R. Enget at Bata - Dokolo Rd
81273	R. Lwanda at Kyotera - Rakai Rd
84207	L. George at Kasenyi
84215	R. Mpanga at Fort Portal - Ibanda
84227	R. Chambura at Kichwamba
84228	R. Nyamugasani at Katwe - Zaire Rd
84267	R. Mitano at Kanungu - Rwensama Rd
87221	R. Albert Nile at Laropi (87221)
87222	R. Albert Nile at Panyango. (87222)
82239	R. Longiro - Near Kotido
84251	L. Bunyonyi at Bwama Island
85205	R. Semliki at Bweramule
85214	R. Wambabya at Buseruka
86201	R. Aswa I at Puranga
86202	R. Aswa II at Gulu - Kitgum Rd
86212	R. Pager at Kitgum
86213	R. Agago at Kitgum - Lira Rd
87207	R. Ayugi at Atiak - Laropi Rd

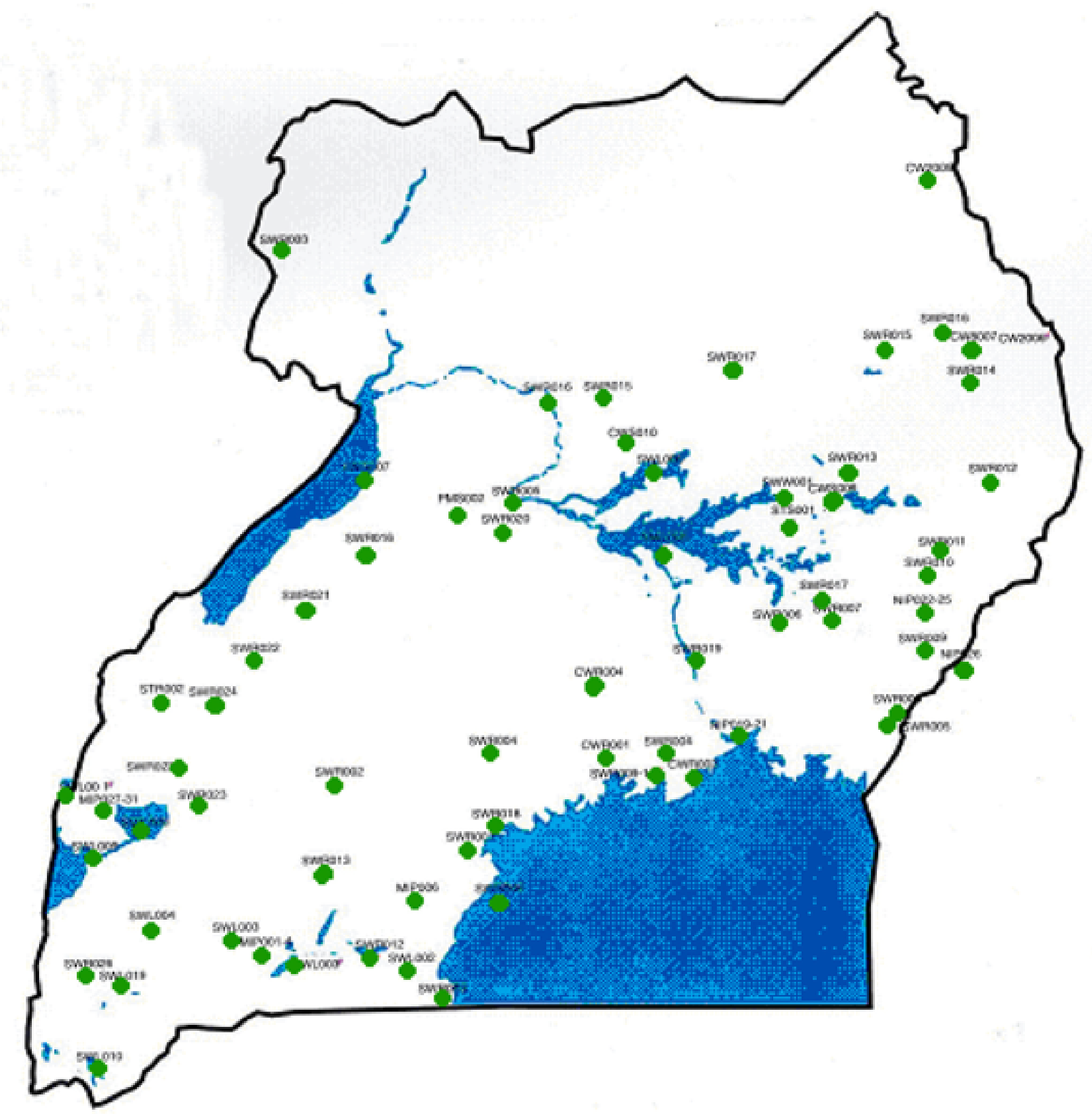
Appendix 2b: Groundwater Monitoring Network in Uganda



Source: UNWD Report, 2005

Groundwater Monitoring Stations		
Station	**Type Of Aquifer**	**Monitoring Purpose**
Nkokonjeru	Bedrock	Monitor effects of production borehole (groundwater abstraction)
002 – Pallisa (Asera Home)	Bedrock	Monitor regional change/ seasonal groundwater fluctuation
003 – Kangole (Moroto)	Bedrock	Monitor effects of production borehole (groundwater abstraction)
004 – Moroto prison	Bedrock	Monitor effects of production borehole (groundwater abstraction)
005 – Bombo Barracks	Regolith and bedrock	Monitor effects of production borehole (groundwater abstraction)
006 - Hoima Hospital	Regolith and bedrock	Monitor effects of production borehole (groundwater abstraction)
007 - Rakai (Civic centre)	Regolith and bedrock	Monitor regional change / seasonal groundwater fluctuation
008 – Lyantonde (Kyabazara)	Regolith and Bedrock	Monitor regional change / seasonal groundwater fluctuation
009 - Mbarara UNICEF Camp	Bedrock	Monitor regional change / seasonal groundwater fluctuation
010 - Rukungiri	Bedrock	Monitor effects of production borehole (groundwater abstraction)
013 - Apac-DWD Offices	Regolith	Monitor regional change / seasonal groundwater fluctuation
012 - Apac-Loro CPAR offices)	Regolith	Monitor regional change / seasonal groundwater fluctuation
014 - Soroti	Bedrock	Monitor regional change / seasonal groundwater fluctuation
015 - Nkozi University	Regolith and bedrock	Monitor effects of production borehole (groundwater abstraction)
016 – WRMD (Entebbe)	Bedrock	Monitor regional change/ seasonal groundwater fluctuation

Appendix 2c: Water Quality Monitoring Network in Uganda



Source: UNWD Report, 2005

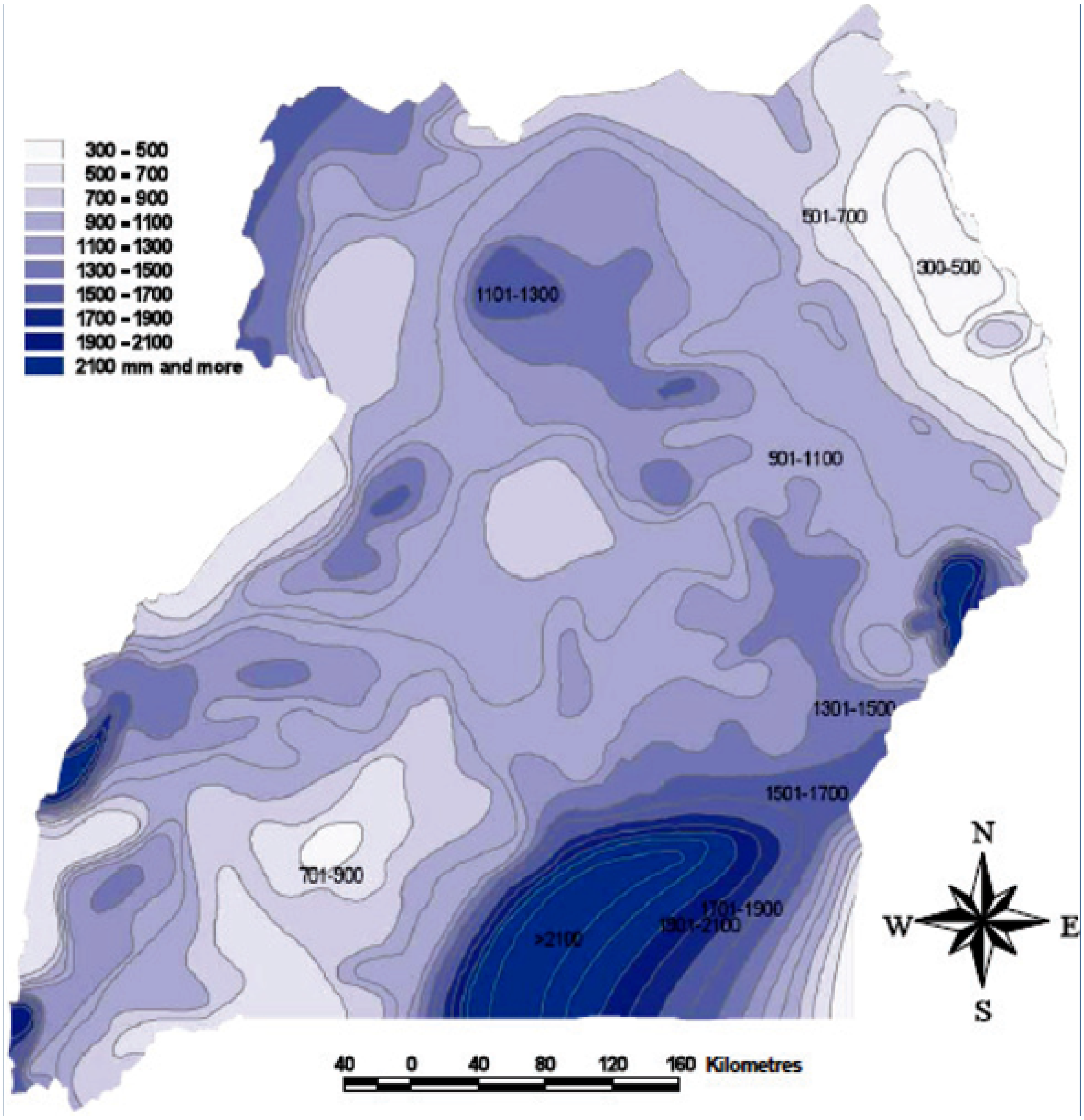
Water Quality Monitoring Stations	
Site_Id	**Site Name**
MIP011	Natete Stream
MIP017	Kinawataka swamp
PWS004	Nyaruzinga Water Works - Bushenyi(Intake)
PWS004A	Nyaruzinga Water Works - Bushenyi(Treated Water)
STS002A	Fort Portal Sewerage Works (After discharge)
GS 1943	Apac Hospital Borehole
GWS001	Port Bell Ground Water Site at PortBell - Kampala
GWS002	Nkokonjeru Ground water Site - Mukono
GWS003	Mbarara Ground Water Site at UNICEF Camp - Mbarara
GWS004	Bombo Ground Water Site at Bombo Army Barracks - Luwero
GWS005	Busia Ground Water Site
GWS006	Soroti Ground Water Site at DWD Camp,Otuchopi - Soroti
GWS007	Morulinga Ground Water Site at Kangole - Moroto
GWS008	Moroto Ground Water Site at Prison Barracks - Moroto
GWS009	Kabong Ground Water Site at Kabong Hospital - Kotido
GWS010	Apac Ground Water Site at DWD Offices - Apac
GWS012	Rakai Ground Water Site at Civic Centre - Rakai
GWS013	Lyantonde Ground Water Site at Kyabazala, Lyantonde - Rak
GWS014	Kasese Ground Water Site at Kasese Cobalt Co. Ltd - Kases
GWS015	Loro Ground Water Site at CPAR Tree Nursery, Loro - Apac
GWS016	Hoima Ground Water Site at Hoima Hospital - Hoima
GWS017	Osera Ground Water Site - Pallisa
GWS018	Nkozi Ground Water Site - Mpigi
GWS019	Rukungiri Ground Water Site - Rukungiri
MIP001	R. Ruizi at NWSC New Water Treatment Works
MIP002	R. Ruizi at NWSC Old lagoons
MIP002A	R. Ruizi at NWSC Lagoons(After discharge)
MIP003	NWSC Lagoons - Mbarara (Effluent Discharge)
MIP004	R. Ruizi Downstream of FREBA Tennery
MIP005	Nakayiba stream at NWSC Lagoons (Before discharge)
MIP005A	Nakayiba stream at NWSC Lagoons (After discharge)
MIP006	Nakayiba stream at Nyendo - Masaka road

Water Quality Monitoring Stations	
Site_Id	**Site Name**
MIP007	Nakayiba at Mbarara by Pass (Kyakumpi)
MIP008	Masaka NWSC Lagoons(Effluent Discharge)
MIP009	Nakivubo Channel (Bridge over Port Bell Railway Line)
MIP010	Nalukolongo Channel
MIP012	Bwaise stream (Upstream)
MIP013	Bwaise stream (Downstream)
MIP014	Lugogo Channel
MIP015	Kitante stream
MIP016	Bat Valley stream (Spring)
MIP018	Kyambogo stream
MIP019	L. Victoria at Kirinya Bay opposite NWSC Lagoons
MIP020	L. Victoria at Masese
MIP021	R. Nile at Owen Falls Bridge
MIP022	Mbale NWSC Old Lagoons (Effluent Discharge)
MIP023	Mbale NWSC New Lagoons (Point of Confluence)
MIP024	Mbale Soap Works (Railway Bridge)
MIP025	Mbale Soap Works (Up Stream)
MIP026	River Lwakhaka (Road Bridge Kenya - Uganda Border)
MIP027	R. Nyamwamba at Kasese - Kilembe road
MIP028	R. Rukoki at Kasese - F/Portal road
MIP029	R. Rukoki at Kasese - Kampala Railway Bridge(Before discharge)
MIP029A	R. Rukoki at Kasese - Kampala Railway Bridge (After discharge)
MIP030	R. Sebwe at Kasese - F/Portal road
MIP031	R. Rukoki (KCCL Effluent)
PWS001	Soroti Water Works(intake)
PWS001A	Soroti Water Works (Treated Water)
PWS001B	Soroti Water Works (Distribution)
PWS002	Masindi Water Works (Intake)
PWS002A	Masindi Water Works (Treated Water)
PWS002B	Masindi Water Works (Distribution)
PWS003	Arua Water Works (Intake)
PWS003A	Arua Water Works (Treated Water)

Water Quality Monitoring Stations	
Site_Id	**Site Name**
PWS003B)	Arua Water Works (Distribution
PWS004B	Nyaruzinga Water Works – Bushenyi (Distribution)
SWR027	R. Ruizi NWSC Treated water reservoir
STS001	Soroti Sewerage Works (Influent)
STS001A	Soroti Sewerage Works (Effluent)
STS002	Fort Portal Sewerage Works (Effluent)
STS002B	Fort Portal Sewerage Works (Before discharge) R. Mugunu
SWL001	L. Nabugabo at Green View
SWL002	L. Kijjanibarora at Kibona Village
SWL003	L. Nakivali at Rukinga Fishing Village
SWL004	L. Wamala at Kitinika Fishing Village
SWL005	L. Kyoga at Bukungu
SWL006	L. Kwania at Nabyeso
SWL007	L. Albert at Butiaba
SWL008	L . George at Kasenyi
SWL009	L. Edward at Katwe
SWL010	L. Bunyonyi at Kyabahinga
SWR001	R. Katonga at Kampala - Masaka road
SWR002	R. Katonga at Kabamba - Nkonge road
SWR003	R. Bukoora at Kasensero road
SWR004	R. Sezibwa at Sezibwa Falls
SWR005	R. Nile at Masindi Port
SWR006	R. Lumbuye at Kaliro - Nawaikoke road
SWR007	R. Mpologoma at Budumba
SWR008	R. Malaba at Busitema
SWR009	R. Manafwa at NWSC Treatment Works
SWR010	R. Simu at Mbale - Moroto road
SWR011	R. Sipi at Mbale - Moroto road
SWR012	R. Namalu at Mbale - Moroto road
SWR013	R. Olumot at Soroti - Moroto road
SWR014	R. Omaniman at Kangole Trading Centre
SWR015	R. Lokorimoru at Nyakwai - Matany road

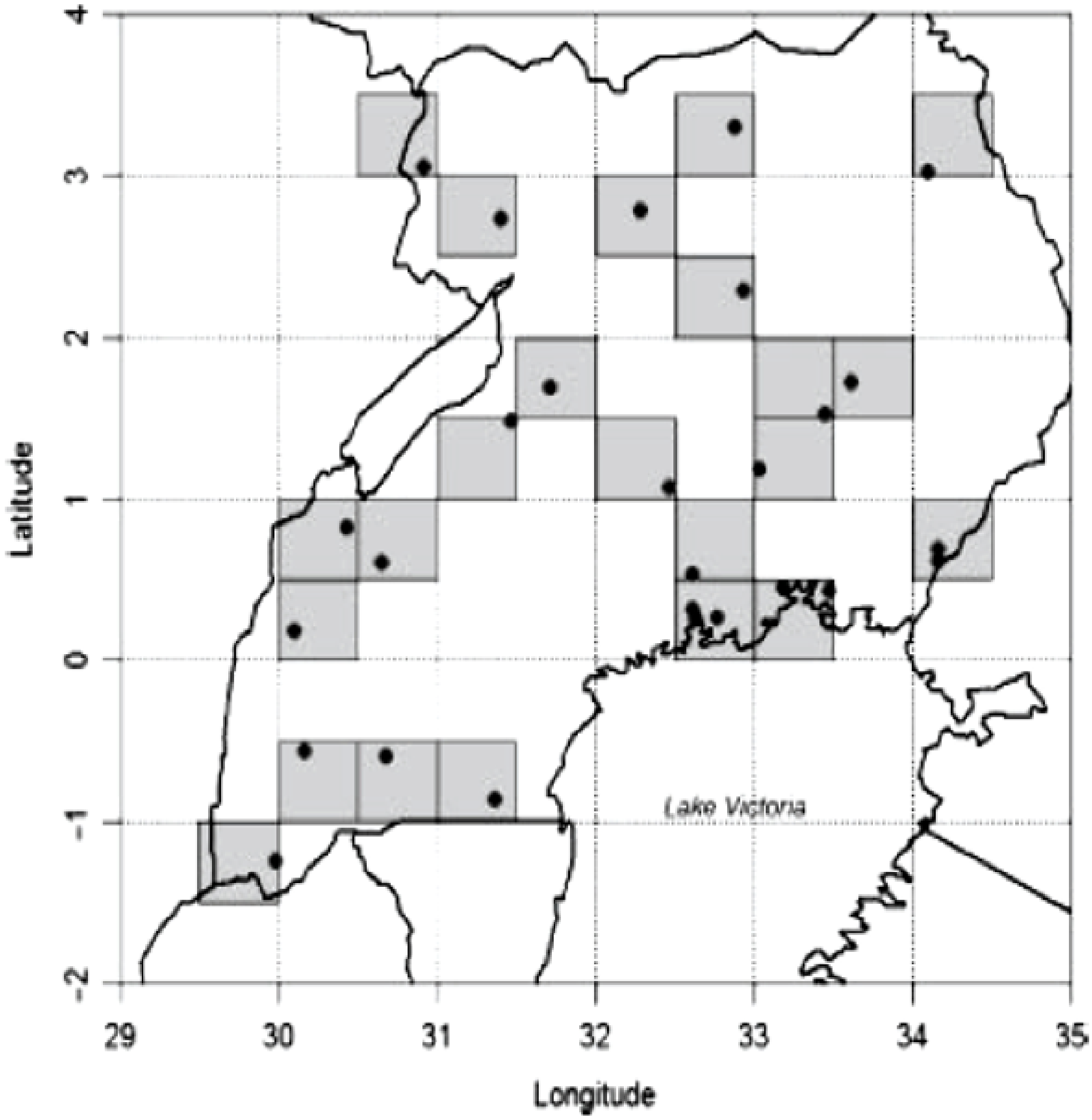
Water Quality Monitoring Stations	
Site_Id	**Site Name**
SWR016	R. Alamacha at Lopei
SWR017	R. Moroto at Aloi - Adwari road
SWR018	R. Tochi at Lira - Kamdini road
SWR019	R. Nile at Mbulamuti Cable Way
SWR020	R. Kafu at Kampala - Gulu road
SWR021	R. Nkuzi at F/Portal - Hoima road
SWR022	R. Muzizi at F/Portal - Hoima road
SWR023 road	R. Mpanga at Kamwenge - Ibanda
SWR024	R. Mpanga at Mubende - Fortportal road
SWR025	R. Mubuku at F/Portal - Kasese road
SWR026	R. Mitano/Mirara at Rukungiri - Kambuga road
SWL010	L. Bunyonyi at Kyabahinga

Appendix 2d: Rainfall Distribution in Uganda



Source: Uganda Atlas (2009

Appendix 2e: Rain Gauge Locations in Uganda



APPENDIX 3: WORLD HEALTH ORGANISATION GUIDELINES ON DRINKING WATER QUALITY

Appendix 3a:	Guideline values for chemicals from industrial sources and human dwellings that are of health significance in drinking-water
Appendix 3b:	Guideline values for pesticides that were previously used for health purposes and are of health significance in drinking-water
Appendix 3c:	Guideline values for naturally occurring chemicals of health significance in drinking-water
Appendix 3d:	Guideline values for chemicals used in water treatment or materials in contact with drinking water that are of health significance in drinking-water
Appendix 3e:	International Organization for Standardization (ISO) standards for detection and Enumeration of Faecal Coliforms.
Appendix 3f:	WHO (2011) Guideline values for verification of microbial quality of drinking water
Appendix 3g:	Maximum Microbiological Limits of Drinking Water
Appendix 3h:	Guidelines for Distributed Water
Appendix 3i:	Requirements for Naturally Occurring Chemicals
Appendix 3j:	Requirements for Chemicals used in Water Treatment[2]
Appendix 3k:	Requirements for Residues of Agricultural Chemicals
Appendix 3l:	Requirements for Chemicals from Industrial Sources
Appendix 3m:	Requirements for Radioactive Matter in Drinking Water
Appendix 3n:	Requirements Affecting Organoleptic and Physical Characteristics
Appendix 3o:	Parameters for Surveillance of Drinking Water Safety
Appendix 3p:	Recommended Parameters for Actual Monitoring with Respective Frequency and Key Sampling Sites
Appendix 3q:	Sampling for Rural Supplies
Appendix 3r:	Guideline values for chemicals from agricultural activities that are of health significance in drinking-water
Appendix 3s:	Disinfection by-products present in disinfected water
Appendix 3t:	Inorganic Constituents of Health significance
Appendix 3u:	Organic Constituents of Health significance
Appendix 3v:	Desirable aesthetic quality

Appendix 3a: Guideline values for chemicals from industrial sources and human dwellings that are of health significance in drinking-water

	Guideline value		
Chemical	**µg/l**	**mg/l**	**Remarks**
Inorganic			
Cadnium	3	0.003	
Mecury	6	0.006	
Boron	2400	2.4	
Organic			
Benzene	10[a]	0.01[a]	
Carbon tetrachloride	4	0.004	
1,2-Dichlorobenzene	1000 (C)	1 (C)	
1,4-Dichlorobenzene	300 (C)	0.3 (C)	
1,2-Dichloroethane	30[a]	0.03[a]	
1,2-Dichloroethene	50	0.05	
Dichloromethane	20	0.02	
Di(2-ethylhexyl)phthalate	8	0.008	
1,4-Dioxane	50[a]	0.05[a]	Derived using TDI approach as well as linear multistage modelling
Edetic acid	600	0.6	Applies to the free acid
Ethylbenzene	300 (C)	0.3 (C)	
Hexachlorobutadiene	0.6	0.0006	
Nitrilotriacetic acid	200	0.2	
Pentachlorophenol	9[a] (P)	0.009[a] (P)	
Styrene	20 (C)	0.02 (C)	
Tetrachloroethene	40	0.04	
Toluene	700 (C)	0.7 (C)	
Trichloroethene	20 (P)	0.02 (P)	
Xylenes	500 (C)	0.5 (C)	

C, concentrations of the substance at or below the health-based guideline value may affect the appearance, taste or odour of the water, leading to consumer complaints; P, provisional guideline value because of uncertainties in the health database

[a]For non-threshold substances, the guideline value is the concentration in drinking-water associated with an upper bound excess lifetime cancer risk of 10^{-5} (one additional case of cancer per 100 000 of the population ingesting drinking-water containing the substance at the guideline value for 70 years). Concentrations associated with estimated upper-bound excess lifetime cancer risks of 10^{-4} and 10^{-6} can be calculated by multiplying and dividing, respectively, the guideline value by 10.

Appendix 3b: Guideline values for pesticides that were previously used for health purposes and are of health significance in drinking-water

Pesticides previously used for public health purposes	Guideline value	
	µg/l	mg/l
DDT and metabolites	1	0.001

Appendix 3c: Guideline values for naturally occurring chemicals of health significance in drinking-water

	Guideline value		
Chemical	**µg/l**	**mg/l**	**Remarks**
Inorganic			
Arsenic	10 (A, T)	0.01 (A, T)	
Barium	700	0.7	
Boron	2400	2.4	
Chromium	50 (P)	0.05 (P)	For total chromium
Fluoride	1500	1.5	Volume of water consumed and intake from other sources should be considered when setting national standards
Selenium	40 (P)	0.04 (P)	
Uranium	30 (P)	0.03 (P)	Only chemical aspects of uranium addressed
Organic			
Microcystin-LR	1 (P)	0.001 (P)	For total microcystin-LR (free plus cell-bound)

A, provisional guideline value because calculated guideline value is below the achievable quantification level; P, provisional guideline value because of uncertainties in the health database; T, provisional guideline value because calculated guideline value is below the level that can be achieved through practical treatment methods, source protection, etc.

Appendix 3d: Guideline values for chemicals used in water treatment or materials in contact with drinking water that are of health significance in drinking-water

	Guideline value[a]		
Chemical	**µg/l**	**mg/l**	**Remarks**
Disinfectants			
Chlorine	5000 (C)	5 (C)	For effective disinfection, there should be a residual concentration of free chlorine of ≥ 0.5 mg/l after at least 30 min contact time at pH < 8.0. A chlorine residual should be maintained throughout the distribution system. At the point of delivery, the minimum residual concentration of free chlorine should be 0.2 mg/l.

	Guideline value[a]		
Chemical	**µg/l**	**mg/l**	**Remarks**
Monochloramine	3 000	3	
Sodium	50 000	50	As sodium dichloroisocyanurate
dichloroisocyanurate	40 000	40	As cyanuric acid
	µg/l	**mg/l**	
Disinfection by-products			
Bromate	10[a] (A, T)	0.01[a] (A, T)	
Disinfection by-products			
Bromate	10[a] (A, T)	0.01[a] (A, T)	
Bromodichloromethane	60[a]	0.06[a]	
Bromoform	100	0.1	
Chlorate	700 (D)	0.7 (D)	
Chlorite	700 (D)	0.7 (D)	
Chloroform	300	0.3	
Dibromoacetonitrile	70	0.07	
Dibromochloromethane	100	0.1	
Dichloroacetate	50 [a] (D)	0.05 [a] (D)	
Dichloroacetonitrile	20 (P)	0.02 (P)	
Chemical	**µg/l**	**mg/l**	**Remarks**
Monochloroacetate	20	0.02	
N-Nitrosodimethylamine	0.1	0.0001	
Trichloroacetate	200	0.2	
2,4,6-Trichlorophenol	200 [a] (C)	0.2 [a] (C)	
Trihalomethanes			The sum of the ratio of the concentration of each to its respective guideline value should not exceed 1
Contaminants from treatment chemicals			
Acrylamide	0.5a	0.0005a	
Epichlorohydrin	0.4 (P)	0.0004 (P)	
Contaminants from pipes and fittings			
Antimony	20	0.02	
Benzo[*a*]pyrene	0.7[a]	0.0007[a]	

Chemical	Guideline value[a] µg/l	mg/l	Remarks
Copper	2000	2	Staining of laundry and sanitary ware may occur below guideline value
Lead	10 (A, T)	0.01 (A, T)	
Nickel	70	0.07	
Vinyl chloride	0.3 [a]	0.0003 [a]	

A, provisional guideline value because calculated guideline value is below the achievable quantification level; C, concentrations of the substance at or below the health-based guideline value may affect the appearance, taste or odour of the water, leading to consumer complaints; D, provisional guideline value because disinfection is likely to result in the guideline value being exceeded; P, provisional guideline value because of uncertainties in the health database; T, provisional guideline value because calculated guideline value is below the level that can be achieved through practical treatment methods, source control, etc.

[a] For substances that are considered to be carcinogenic, the guideline value is the concentration in drinking-water associated with an upper-bound excess lifetime cancer risk of 10−5 (one additional case of cancer per 100 000 of the population ingesting drinking-water containing the substance at the guideline value for 70 years). Concentrations associated with estimated upper-bound excess lifetime cancer risks of 10−4 and 10−6 can be calculated by multiplying and dividing, respectively, the guideline value by 10.

Appendix 3e: International Organization for Standardization (ISO) standards for detection and Enumeration of Faecal Coliforms.

ISO standard	Title (water quality)
6461-1:1986	Detection and enumeration of the spores of sulfite-reducing anaerobes (clostridia)—Part 1: Method by enrichment in a liquid medium
6461-2:1986	Detection and enumeration of the spores of sulfite-reducing anaerobes (clostridia)—Part 2: Method by membrane filtration
7704:1985	Evaluation of membrane filters used for microbiological analyses
9308-1:2000	Detection and enumeration of *Escherichia coli* and coliform bacteria—Part 1: Membrane filtration method
9308-2:1990	Detection and enumeration of coliform organisms, thermo-tolerant coliform organisms and presumptive *Escherichia coli*—Part 2: Multiple tube (most probable number) method
9308-3:1998	Detection and enumeration of *Escherichia coli* and coliform bacteria—Part 3: Miniaturized method (most probable number) for the detection and enumeration of *E. coli* in surface and waste water
10705-1:1995	Detection and enumeration of bacteriophages—Part 1: Enumeration of F-specific RNA bacteriophages
10705-2:2000	Detection and enumeration of bacteriophages—Part 2: Enumeration of somatic coliphages
10705-3:2003	Detection and enumeration of bacteriophages—Part 3: Validation of methods for concentration of bacteriophages from water
10705-4:2001	Detection and enumeration of bacteriophages—Part 4: Enumeration of bacteriophages infecting *Bacteroides fragilis*

Appendix 3f: WHO (2011) Guideline values for verification of microbial quality of drinking water

Organisms	Guideline value
All water directly intended for drinking[a] E. coli or thermo-tolerant coliform bacteria	Must not be detectable in any 100ml sample
Treated water entering the distribution system[b] E .coli or thermo-tolerant coliform bacteria	Must not be detectable in any 100ml sample
Treated water in the distribution system[c] E. coli or thermo-tolerant coliform bacteria	Must not be detectable in any 100ml sample

[a]Immediate investigation if *E.coli* are detected.
[b]Although *E.coli* are a more precise indicator of faecal pollution, thermo-tolerant coliform is an alternative. If necessary confirmatory tests should be carried out. Total coliform bacteria are not acceptable indicators of the sanitary quality of water supplies, particularly in tropical areas where many bacteria of no sanitary significance occur in almost all untreated water supplies.
[c]It is recognized that in the great majority of rural water supplies especially in developing countries faecal contamination is widespread. Especially under these conditions, medium term targets for the progressive improvement of water supplies should be set.

Appendix 3g: Maximum Microbiological Limits of Drinking Water

Microorganism	**Allowable compliance limits and contribution of samples, % a)**			**Method of test**
	Minimum 95%	Maximum of 4% of samples	Maximum of 1% of samples	
Heterotrophic plate count, count/mL	100	1 000	10 000	US ISO 6222
Total coliform bacteria count b), count/100 mL	Not detected	10	100	US ISO 9308-2
E. coli, count b),/100 mL	Not detected	Not detected	1	
Clostridium perfringens (including viable spores), count/100 mL	Not detected	Not detected	Not detected	Annex A
Somatic coliphages count, count/10 mL	Not detected	1	10	US ISO 10705-2
Enteric viruses count, count/100 L	Not detected	1	10	Annex A
Protozoan parasites *(Giardia/ Cryptosporidium)* count, count/ 10 L	Not detected	Not detected	1	US ISO 15553

a), The allowable compliance contribution shall be at least 95 % of the samples to conform to the limits of 100 CFU/mL, and a maximum of 4 % and 1 % of samples, respectively, to conform to the limits of 1000 CFU/mL and 10 000 CFU/mL. The objective of disinfection should, nevertheless, be to attain 100 % compliance to the limit of 100 CFU/100mL.
b), In most instances it will not be necessary to conduct both these tests; one or the other will normally suffice as the required indicator.

Appendix 3h: Guidelines for Distributed Water

Parameter	Unit	Guideline Value	Remarks
A. PIPED WATER SUPPLIES			
A.1 Treated water entering the distribution system			
Faecal Coliforms Coliform Organisms	Number/ 100ml Number/ 100ml	Zero Zero	Turbidity < 1 NTU; for disinfection with chlorine. pH preferably<8.0. free chlorine residual 0.2-0.5 mg/l following 33 minutes (minimum) contact
A.2 Untreated water entering the distribution system			
Faecal coliforms	Number/ 100ml	Zero	
Coliform Organisms	Number/ 100ml	Zero	In 98% of samples examined throughout the year for large supplies with sufficient samples examined.
Coliform Organisms	Number/ 100ml	3	In occasional sample but not in consecutive samples
A.3 Water in distribution system			
Faecal coliforms	Number/ 100ml	Zero	
Coliform Organisms	Number/ 100ml	Zero	In, 95% of samples examined throughout the year for large supplies with sufficient samples examined.
Coliform Organisms	Number/ 100ml	3	In occasional samples but not in consecutive samples.
B. UNPIPED WATER SUPPLIES			
Faecal Coliforms	Number/ 100ml	Zero	
Coliform Organisms	Number/ 100ml	10	Not occurring repeatedly. Repeated occurrence and failure to improve sanitary protection, alternate source to be found if possible

Appendix 3i: Requirements for Naturally Occurring Chemicals

Chemical	Maximum limit, mg/L		Method of test
	Class I	Class II	
Arsenic	0.01 calculated as total As	0.05	FDUS ISO 11969
Barium	0.7	1.0	Annex A
Boron	1.0		US ISO 9390,
Chromium	0.05 calculated as total Cr.	0.05	US ISO 9174
Fluoride	1.0	1.5	US ISO 10359-2
Manganese	1.0	0.1	US ISO 6333
Molybdenum	0.07		Annex A
Selenium	0.01	0.01	US ISO 9965
Uranium	0.015	0.015	Annex A
Mercury	0.001	0.001	US ISO 16590

Appendix 3j: Requirements for Chemicals used in Water Treatment2

Chemical	Required limit, max, mg/L	Method of test
Acrylamide	0.0001	*
Antimony	0.005	Annex A
Benzo[a]pyrene	0.0001	*
Bromate	0.01	US ISO 15061
Bromodichloromethane	0.06	*
Bromoform	0.1	*
Chlorate	0.7	*
Chlorine	5	US ISO 7393-1
Chlorite	0.7	*
Chloroform	0.3	*
Copper	2	US ISO 8288
Cyanogen chloride	0.07	US ISO 6703-3
Dibromoacetonitrile	0.07	*
Dibromochloromethane	0.1	*
Dichloroacetate	0.05	*
Epichlorohydrin	0.0001	*
Lead	0.01	US ISO 8288
Monochloroacetate	0.02	*
Monochloroamine	3	*
Nickel	0.02	US ISO 8288
Trichloroacetate	0.2	*
Trichlorophenol, 2,4,6-	0.2	*
Trihalomethanes[3], total	0.1	*
Vinyl chloride	0.0003	*
* Methods of test of the ASTM or APHA may be used (see Annex A)		

2) Includes disinfectants, disinfections by products, contaminants from treatment chemicals and contaminants from pipes and fittings

3) Trihalomethanes (bromoform, bromodichloromethane, dibromochloromethane, chloroform) are formed in drinking-water primarily as a result of chlorination of organic matter present naturally in raw water supplies.

Appendix 3k: Requirements for Residues of Agricultural Chemicals

Chemical	Maximum limit, mg/L, Max.	Remarks
Alachlor	0.02	**a)**
Aldicarb	0.01	Applies to aldicarb sulfoxide and aldicarb sulfone
Aldrin and dieldrin	0.00003	For combined aldrin plus dieldrin
Atrazine	0.002	**a)**
Chlordane	0.0002	Currently 0.0003
Chlorpyrifos[b)]	0.03	**a)**
Chlorotoluron	0.03	**a)**
2,4-D (2,4-dichlorophenoxyacetic	0.03	Applies to free acid acid)
2,4-DB	0.09	**a)**
DDT and metabolites [b)]	0.001	**a)**
Dichlorprop	0.1	**a)**
Dimethoate	0.006	**a)**
Endrin	0.0006	**a)**
Fenoprop	0.009	**a)**
Isoproturon	0.009	**a)**
Lindane	0.002	**a)**
MCPA	0.002	**a)**
Mecoprop	0.01	**a)**
Methoxychlor	0.02	**a)**
Metolachlor	0.01	**a)**
Molinate	0.006	**a)**
Nitrate (as NO3)	50	Short-term exposure
Nitrite (as NO2)	3	Short-term exposure
	0.2	Long-term exposure
Permethrin [b)]	0.3	Only when used as a larvicide for
Pyriproxyfen [b)]	0.3	**a)**
Simazine	0.002	**a)**
2,4,5-T	0.009	**a)**
Terbuthylazine	0.007	**a)**

a) Methods of test of the ASTM or APHA may be used (see Annex A)
b) Chemicals used in water for public health purposes such as treatment of mosquito nets and larvicidal effects on water surfaces

Appendix 3l: Requirements for Chemicals from Industrial Sources

Chemical	Required limit, mg/L , max	Method of test
Benzene	0.001	US ISO 11423-1 US ISO 11423-2
Cadmium	0.003	US ISO 5961 US ISO 8288
Carbon tetrachloride	0.004	*
Cyanide	0.07	US ISO 14403
Di(2-ethylhexyl)phthalate	0.008	*
Dichlorobenzene, 1,2-	1	*
Dichlorobenzene, 1,4-	0.3	*
Dichloroethane, 1,2-	0.03	*
Dichloroethene, 1,2-	0.05	*
Dichloromethane	0.02	*
Dioxane, 1,4-	0.05	*
Edetic acid (EDTA)	0.6	Applies to the free acid
Ethylbenzene	0.3	*
Hexachlorobutadiene	0.0006	*
Nitrilotriacetic acid (NTA)	0.2	*
Pentachlorophenol	0.009	*
Styrene	0.02	*
Tetrachloroethene	0.04	*
Toluene	0.7	*
Trichloroethene	0.02	*
Xylenes	0.5	*
* Methods of test of the ASTM or APHA may be used (see Annex A)		

Appendix 3m: Requirements for Radioactive Matter in Drinking Water

Total radioactivity present in the form of	Maximum limit
Total beta activity (except K40 and H3) *	1 Bq/l
Total alpha activity	0.5 Bq/l
* The contribution of potassium-40 to beta activity shall be calculated using beta activity of 27.6 Bq/g of stable potassium and subtracted from total beta activity. If beta activity levels are higher than 1 Bq/l after adjusting for potassium-40 beta activity due to tritium shall be determined and shall not exceed 100 Bq/l. A Becquerel (Bq) is a unit of radioactivity.	

Appendix 3n: Requirements Affecting Organoleptic and Physical Characteristics

Characteristic	Requirement levels		Method of test
	Class I	Class II	
Colour	15 true colour units (TCU)	15 true colour units (TCU)	US ISO 7887
Taste	Acceptable to consumers and no abnormal changes	Acceptable to consumers and no abnormal changes	-
Odour	Acceptable to consumers and no abnormal changes	Acceptable to consumers and no abnormal changes	Annex A
Electrical conductivity, at 25°C	1500 µS/cm	2500 µS/cm	US ISO 7888
pH	5.5 – 8.5	6.5 – 8.5	US ISO 10523
Turbidity	5 Nephelometric Turbidity Units	10 Nephelometric Turbidity Units	US ISO 7027
Total dissolved solids (TDS)	500 mg/L	1 500 mg/L	Annex A
Iron	0.2 mg/L	1 mg/L	US ISO 6332,
Ammonia	0.5 mg/L	1.0 mg/L	US ISO 5664
Aluminium	0.2 mg/L	0.2 mg/L	US ISO 10566 US ISO 12020
Potassium as K, mg/l, max.	50	100	US ISO 9964-1
Sodium as Na, mg/l, max.	200 mg/L	400 mg/L	US ISO 9964-1
Chloride	250 mg/L	500 mg/L	US ISO 10304-1
Magnesium	100 mg/L	150 mg/L	US ISO 6058, US ISO 6059 US ISO 7980

Appendix 3o: Recommended Parameters for Actual Monitoring with Respective Frequency and Key Sampling Sites

Parameters	Raw water intake: Surface water or Groundwater	Production	At supply area/ distribution Networks
PHYSIO-CHEMICAL PARAMETERS			
Appearance	✓	✓	✓
Smell/odour	✓	✓	✓
Taste	✓	✓	✓
pH	✓	✓	✓
Temperature	✓	✓	✓
Electrical conductivity	✓	✓	✓
Colour	✓	✓	✓

Parameters	Raw water intake: Surface water or Groundwater	Production	At supply area/ distribution Networks
Turbidity	✓	✓	✓
Dissolved oxygen	✓		
Dissolved carbon dioxide	✓		
Alkalinity (phenolphthalein)	✓	✓	✓
Alkalinity (Total)	✓	✓	✓
Hardness	✓	✓	✓
Chloride	✓	✓	✓
Fluoride	✓	✓	✓
Sulphate	✓	✓	✓
Carbonate	✓		
Aluminium		✓	✓
Iron	✓	✓	✓
Manganese	✓	✓	✓
Nitrogen-ammonia	✓		✓
Nitrogen-nitrate	✓	✓	✓
Phosphate	✓		✓
Chlorine: free		✓	✓
Chlorine: Total		✓	✓
BIOLOGICAL AND MICROBIOLOGICAL PARAMETERS			
Coliform: faecal	✓	✓	✓
Coliform: Total			✓
Sampling frequency	daily (SW, monthly GW)	daily (physio-chemical and microbiological) to quarterly (toxicants)	weekly to fortnightly (densely populated)
Depending on circumstances at hand such a parameter may be analysed			
Algae	✓	✓	✓
Chlorophyll	✓	✓	✓
TOXICANT PARAMETERS			
Lead	✓	✓	✓
Copper	✓	✓	✓
Zinc		✓	✓
Arsenic		✓	✓
Sampling frequency	weekly till concentration confirmed to be acceptable or source eliminated		

Appendix 3p: Sampling for Rural Supplies

Technology	Frequency	
	Dry season	**Rainy season**
Deep boreholes	Monthly	Monthly
Shallow Wells	Monthly	Fortnightly
Springs	Monthly	Weekly
Gravity Flow Scheme (GFS)	Monthly	Fortnightly
Rain water	Monthly	Weekly

Appendix 3q: Guideline values for chemicals from agricultural activities that are of health significance in drinking-water

Guideline value			
Chemical	**µg/l**	**mg/l**	**Remarks**
Non-pesticides			
Nitrate (as NO_3^-)	50 000	50	Short-term exposure
Nitrite (as NO_2^-)	3 000	3	Short-term exposure; a provisional guideline value for chronic effects of nitrite that was in the third edition has been suspended and is under review owing to significant uncertainty surrounding the endogenous formation of nitrite and concentrations in human saliva.
Pesticides used in agriculture			
Alachlor	20[a]	0.02[a]	
Aldicarb	10	0.01	Applies to aldicarb sulfoxide and aldicarb sulfone
Aldrin and dieldrin	0.03	0.000 03	For combined aldrin plus dieldrin
Atrazine and its chloro-striazine metabolites	100	0.1	
Carbofuran	7	0.007	
Chlordane	0.2	0.0002	
Chlorotoluron	30	0.03	
Chlorpyrifos	30	0.03	
Cyanazine	0.6	0.0006	
2,4-D[b]	30	0.03	Applies to the free acid
2,4-DB[c]	90	0.09	
1,2-Dibromo-3-chloropropane	1[a]	0.001[a]	
1,2-Dibromoethane	0.4[a](P)	0.0004[a](P)	

Guideline value			
Chemical	µg/l	mg/l	Remarks
1,2-Dichloropropane	40(P)	0.04(P)	
1,3-Dichloropropene	20[a]	0.02[a]	
Dichlorprop	100	0.1	
Dimethoate	6	0.006	
Endrin	0.6	0.0006	
Fenoprop	9	0.009	
Hydroxyatrazine	200	200 0.2	Atrazine metabolite
Isoproturon	9	0.009	
Lindane	2	0.002	
MCPA[d]	2	0.002	
Mecoprop	10	0.01	
Methoxychlor	20	0.02	
Metolachlor	10	0.01	
Molinate	6	0.006	
Pendimethalin	20	0.02	
Simazine	2	0.002	
2,4,5-T[e]	9	0.009	
Terbuthylazine	7	0.007	
Trifluralin	20	0.02	

P, provisional guideline value because of uncertainties in the health database

[a]For substances that are considered to be carcinogenic, the guideline value is the concentration in drinking-water associated with an upper-bound excess lifetime cancer risk of 10−5 (one additional cancer per 100 000 of the population ingesting drinking-water containing the substance at the guideline value for 70 years). Concentrations associated with estimated upper-bound excess lifetime cancer risks of 10−4 and 10−6 can be calculated by multiplying and dividing, respectively, the guideline value by 10.

[b]2,4-Dichlorophenoxyacetic acid.

[c]2,4-Dichlorophenoxybutyric acid.

[d]4-(2-Methyl-4-chlorophenoxy) acetic acid.

[e]2,4,5-Trichlorophenoxyacetic acid.

Appendix 3r: Disinfection by-products present in disinfected water

Disinfectant	Significant organohalogen products	Significant inorganic products	Significant non-halogenated products
Chlorine/ hypochlorous acid (hypochlorite	THMs, HAAs, haloacetonitriles, chloral hydrate, chloropicrin, chlorophenols, *N*-chloramines, halofuranones, bromohydrins	Chlorate (mostly from hypochlorite use)	Aldehydes, cyanoalkanoic acids, alkanoic acids, benzene, carboxylic acids, *N*-nitrosodimethylamine
Chlorine dioxide		Chlorite, chlorate	Unknown
Chloramine	Haloacetonitriles, cyanogen chloride, organic chloramines, chloramino acids, chloral hydrate, haloketones	Nitrate, nitrite, chlorate, hydrazine	Aldehydes, ketones, *N*-nitrosodimethylamine
Chloramine	Bromoform, monobromoacetic acid, dibromoacetic acid, dibromoacetone, cyanogen bromide	Chlorate, iodate, bromate, hydrogen peroxide, hypobromous acid, epoxides, ozonates	Aldehydes, ketoacids, ketones, carboxylic acids
Sodium dichloroisocyanurate	As for chlorine/ hypochlorous acid (hypochlorite)		Cyanuric acid

Appendix 3s: Inorganic Constituents of Health significance

Parameter	Unit	Guideline Value	Remarks
Antimony (Sb)	mg/l	0.005	-
Arsenic (As)	mg/l	0.01	
Barium (Ba)	mg/l	0.7	
Beryllium	mg/l	No guideline value set	
Boron (B)	mg/l	0.3	
Cadmium (Cd)	mg/l	0.003	
Chromium (Cr)	mg/l	0.05	
Copper (Cu)	mg/l	2	
Cyanide (Cn)	mg/l	0.07	
Fluoride (F)	mg/l	1.5	Local or climatic conditions may necessitate adaption.
Lead(Pb)	mg/l	0.01	

Manganese (Mn)	mg/l	0.05	
Mercury (Hg)	mg/l	0.001	
Molybdenum (Mo)	mg/l	0.07	
Nickel (Ni)	mg/l	0.02	
Nitrate **(NO_3)**	mg/l	50	
Nitrite ($N0_3$)	mg/l	3	
Selenium (Se)	mg/l	0.01	
Sulphate (SO,)	mg/l	250	
Thallium	-	No guideline value set	
Uranium	-	No guideline value set	

Appendix 3t: Organic Constituents of Health significance

Parameter	Guideline Value
Organics (non-pesticide)	
a. Chlorinated alkanes:	
Carbon tetrachloride	2
Dichloromethane	20
1,1 - dichloroethane	NAD
1,2 - dichloroethane	30
1,1,1-trichloroethane	2000
1,1,2-do	-
b. Chlorinated ethenes.	
Vinyl chloride	5
1,1 - dichloroethene	30
1.2 - dichloroethenes	50
Cis 1.2 - dichloroethene	-
bans - do	-
Trichloroethene	70
Tetrachloroethene	40
c. Aromatic hydrocarbons:	
Benzene	10
Toluene	700
Xylenes	500
ethylbenzene	300
Styrene	20
benzo (a) pyrene	0.7
d. Chlorinated benzenes,	
monochlorobenzene	300

Parameter	Guideline Value
1,2 - dichlorobertzene	1000
1,4 - dichlorobertzene	, 300
1 Trichlorobenzenes (total)	20
1,2,4 - trichlorobenzene	-
e. Miscellaneous Organics	
di (2 - ethylhexy) adipate	80
di (2 - ethythexy) phthalate	8
acrylamide	0.5
epichlorohydrin	0.4
hexachlorobutadiene	0.6
hexachlorocyclopentadiene	-
2,3.7,8-TCDD (dioxin)	-
EDTA	200
nitrilotriacetic acid	200
Dialkyltins	NAD
Tributylin oxide	2
Pesticides	
alachlor	20
aldicarb	10
aldicarb sulfone	-
- do – sulfoxide	-
Aldrin/diedrin	0.03
atrazine	2
bentazon	30
carboofuran	5
chlordane	0.2
chlortoluron	30
DDT	2
1,2 - dibromo - 3 - chloropropane	1
2,4-D	30
1.2- di chloropropane	20
1,3- di chloropropane	NAD
1,3- dichloropropene	20
dalopon	-
dinoseb	-
diquat	-
ethylene dibromide	NAD
endothall	-

Parameter	Guideline Value
endrin	-
glyphosate	-
heptachlor/heptachlor epoxide	0.03
exchlorobenzene	1
isoproturon	9
lindane	2
MCPA	2
methoxychlor	20
metolachlor	10
molinate	6
oxamyl (vydate	-
pendimethalin	20
pentachlorophenol	9
permethrin	20
picloram	-
polychlorinated byphenyls (PCBs)	-
propanil	20
pyridate	100
simazine	2
toxaphene	-
trifluralin	20
chlorophenoxy herbicide other than 2,4-D and MCPA:	
dichlorprop	100
2,4 - DB	90
2,4,5-T	9
silvex	9
mecoprop	10
MCPB	NAD
Disinfectant by-products	
bromate	25
chlorite	200
chlorate	NAD
chlorophenois	200
2.4,6-trichloropheno!	900
formaldehyde	
trihalomethanes (THMs);	
bromoform (tribromomethane)	100
dibromochloro - methane	100

Parameter	Guideline Value
bromodi chloro-methane	60
chloroform (trichloromethane)	200
chlorinated acetic acids;	
dichloroacetic acid	50
trichloroacetic acid	100
trlchloroacetaldetryde/chloral hydrate	10
haloacetonitiries:	
dibromoacetonitrile	100
dichloroacetonitrile	90
trichloroacetonitrile	1
cynogen chloride (as CN”	70

Appendix 3u: Desirable aesthetic quality

Parameter	Guideline Value
Physical Characteristics	
Colour (TCU)	15
Taste and Odour (TON	
Hydrogen-sulphide	0.05 mg/l
pH	<8.0
Temperature	
Tubidity	5 NTU<1 NTU
Total dissolved solids	1000 mg/l
Conductivity	
Oxidisability (Permanganate value)	
Hardness (Ca)	
Substances undesirable in excess (to prevent	
colour, taste, corrosion etc)	
Alkalinity (HCO_3)	
Aluminium	0.2 mg/l
Ammonia	1.5 NH_3, mg/l
Calcium	250 mg/l
Chloride	1 mg/l
Copper	
Iron	0.3 mg/l
Magnesium	0.10 mg/l
Manganese	
Phenols (c_6H_5OH)	
Phosphorus	
Potassium	
Silver	
Sodium	200 mg/l
Sulphate	250 mg/l
zinc	3 mg/l

APPENDIX 4: ECONOMICS DATA

APPENDIX 4a: ANNUAL CAPITAL COSTS (ANNUITIES)

APPENDIX 4b: PRESENT VALUE FACTORS

APPENDIX 4a: ANNUAL CAPITAL COSTS (ANNUITIES)

Period/discount rate		Discount Factors																			
	1%	2%	3%	4%	5%	6%	7%	8%	9%	10%	11%	12%	13%	14%	15%	16%	17%	18%	19%	20%	25%	30%
1	0.990	0.990	0.974	0.962	0.952	0.943	0.935	0.926	0.917	0.909	0.901	0.893	0.885	0.877	0.870	0.862	0.855	0.847	0.840	0.833	0.800	0.769
2	0.980	0.961	0.943	0.925	0.907	0.890	0.873	0.852	0.842	0.826	0.812	0.797	0.783	0.769	0.756	0.743	0.731	0.718	0.706	0.694	0.640	0.592
3	0.971	0.942	0.915	0.889	0.864	0.840	0.816	0.794	0.772	0.751	0.731	0.712	0.693	0.675	0.658	0.641	0.624	0.609	0.593	0.579	0.512	0.455
4	0.961	0.924	0.888	0.655	0.823	0.792	0.763	0.735	0.708	0.683	0.659	0.636	0.613	0.592	0.572	0.592	0.534	0.516	0.499	0.482	0.410	0.350
5	0.951	0.906	0.863	0.822	0.784	0.747	0.713	0.681	0.650	0.621	0.593	0.567	0.543	0.519	0.497	0.476	0.456	0.437	0.419	0.402	0.328	0.269
6	0.942	0.888	0.837	0.790	0.746	0.705	0.666	0.630	0.596	0.564	0.535	0.507	0.480	0.456	0.432	0.410	0.390	0.370	0.352	0.335	0.262	0.207
7	0.933	0.871	0.813	0.760	0.711	0.665	0.623	0.583	0.547	0.513	0.482	0.452	0.425	0.400	0.376	0.354	0.333	0.314	0.296	0.279	0.210	0.159
8	0.923	0.853	0.789	0.731	0.677	0.627	0.582	0.540	0.502	0.467	0.434	0.404	0.376	0.351	0.327	0.305	0.285	0.266	0.249	0.233	0.168	0.123
9	0.914	0.837	0.766	0.703	0.645	0.592	0.544	0.500	0.490	0.424	0.391	0.361	0.333	0.308	0.284	0.263	0.243	0.225	0.209	0.194	0.134	0.094
10	0.905	0.820	0.744	0.676	0.614	0.558	0.508	0.463	0.422	0.386	0.362	0.322	0.295	0.270	0.247	0.227	0.208	0.191	0.176	0.162	0.107	0.073
11	0.896	0.804	0.722	0.650	0.585	0.527	0.475	0.429	0.388	0.350	0.317	0.287	0.261	0.237	0.215	0.195	0.178	0.162	0.148	0.135	0.096	0.056
12	0.887	0.788	0.701	0.625	0.557	0.497	0.444	0.397	0.356	0.319	0.286	0.257	0.231	0.208	0.187	0.168	0.152	0.137	0.124	0.112	0.069	0.043
13	0.879	0.773	0.681	0.601	0.530	0.469	0.415	0.368	0.326	0.290	0.258	0.229	0.204	0.182	0.163	0.145	0.13	0.116	0.104	0.930	0.055	0.033
14	0.870	0.758	0.661	0.577	0.505	0.442	0.388	0.340	0.299	0.263	0.232	0.205	0.181	0.160	0.141	0.125	0.111	0.099	0.088	0.078	0.044	0.025
15	0.861	0.743	0.642	0.555	0.481	0.417	0.362	0.315	0.275	0.239	0.209	0.183	0.160	0.140	0.123	0.108	0.095	0.084	0.074	0.065	0.035	0.020
16	0.853	0.728	0.623	0.534	0.458	0.394	0.339	0.292	0.252	0.218	0.188	0.163	0.141	0.123	0.107	0.093	0.084	0.071	0.062	0.054	0.028	0.015

Period/discount rate			Discount Factors																		
	1%	2%	3%	4%	5%	6%	7%	8%	9%	10%	11%	12%	13%	14%	15%	16%	17%	18%	19%	20%	25%	30%
17	0.844	0.714	0.605	0.313	0.436	0.371	0.317	0.270	0.231	0.198	0.170	0.146	0.125	0.108	0.093	0.080	0.069	0.060	0.052	0.045	0.023	0.012
18	0.836	0.700	0.587	0.494	0.416	0.350	0.296	0.250	0.212	0.180	0.153	0.130	0.111	0.095	0.081	0.069	0.059	0.051	0.044	0.038	0.018	0.009
19	0.828	0.696	0.557	0.475	0.396	0.331	0.277	0.232	0.194	0.164	0.138	0.116	0.098	0.083	0.070	0.060	0.051	0.043	0.037	0.031	0.014	0.007
20	0.820	0.673	0.554	0.456	0.377	0.312	0.258	0.215	0.178	0.149	0.124	0.104	0.084	0.073	0.061	0.051	0.043	0.037	0.031	0.026	0.012	0.005
21	0.811	0.690	0.538	0.439	0.356	0.294	0.242	0.199	0.164	0.135	0.112	0.093	0.077	0.064	0.053	0.044	0.037	0.031	0.026	0.022	0.009	0.004
22	0.803	0.647	0.522	0.422	0.342	0.278	0.226	0.184	0.150	0.123	0.101	0.083	0.068	0.056	0.046	0.038	0.032	0.026	0.022	0.018	0.007	0.003
23	0.795	0.634	0.507	0.406	0.326	0.262	0.211	0.170	0.138	0.112	0.091	0.074	0.060	0.049	0.047	0.033	0.027	0.022	0.018	0.015	0.006	0.002
24	0.788	0.622	0.492	0.39	0.310	0.247	0.197	0.158	0.126	0.102	0.082	0.066	0.053	0.043	0.035	0.028	0.023	0.019	0.015	0.013	0.005	0.002
25	0.780	0.610	0.478	0.375	0.295	0.233	0.184	0.146	0.116	0.092	0.074	0.059	0.047	0.038	0.030	0.024	0.020	0.016	0.013	0.010	0.004	0.001
26	0.772	0.598	0.464	0.361	0.281	0.220	0.172	0.135	0.106	0.084	0.066	0.053	0.042	0.033	0.026	0.021	0.017	0.014	0.011	0.009	0.003	0.001
27	0.764	0.586	0.450	0.347	0.268	0.207	0.161	0.125	0.096	0.076	0.600	0.047	0.037	0.029	0.023	0.018	0.014	0.011	0.009	0.007	0.002	0.001
28	0.757	0.574	0.437	0.333	0.255	0.196	0.150	0.116	0.090	0.069	0.540	0.042	0.033	0.026	0.020	0.016	0.012	0.010	0.008	0.006	0.002	0.001
29	0.749	0.563	0.424	0.321	0.243	0.185	0.141	0.107	0.082	0.063	0.480	0.037	0.029	0.022	0.017	0.014	0.011	0.008	0.006	0.005	0.002	0.000
30	0.742	0.552	0.412	0.308	0.231	0.174	0.131	0.099	0.057	0.057	0.044	0.033	0.026	0.020	0.015	0.012	0.009	0.007	0.005	0.004	0.001	0.000
31	0.672	0.453	0.307	0.208	0.142	0.097	0.067	0.046	0.032	0.022	0.013	0.011	0.008	0.005	0.004	0.003	0.002	0.001	0.001	0.001	0.000	0.000
32	0.608	0.372	0.228	0.141	0.087	0.054	0.034	0.021	0.013	0.009	0.005	0.003	0.002	0.001	0.001	0.001	0.000	0.000	0.000	0.000	0.000	0.000

Period Discount Rate			Annuity Factors																		
	1%	2%	3%	4%	5%	6%	7%	8%	9%	10%	11%	12%	13%	14%	15%	16%	17%	18%	19%	20%	25%	30%
1	0.990	0.980	0.971	0.962	0.952	0.943	0.935	0.926	0.917	0.909	0.901	0.893	0.885	0.877	0.870	0.862	0.855	0.847	0.840	0.833	0.800	0.769
2	1.970	1.942	1.913	1.886	1.859	1.833	1.808	1.783	1.759	1.736	1.713	1.690	1.668	1.647	1.626	1.605	1.585	1.566	1.547	1.528	1.44	1.361
3	2.941	2.884	2.829	2.775	2.723	2.673	2.624	2.577	2.531	2.487	2.444	2.402	2.361	2.322	2.283	2.246	2.210	2.174	2.140	2.106	1.952	1.816
4	3.902	3.808	3.717	3.630	3.546	3.465	3.387	3.312	3.240	3.170	3.102	3.037	2.974	2.914	2.855	2.798	2.743	2,690	2.639	2.589	2.362	2.166
5	4.853	4.713	4.580	4.452	4.329	4.212	4.100	3.993	3.890	3.791	3.696	3.605	3.517	3.433	3.352	3.274	3.199	3.127	3.058	2.991	2.689	2.436
6	5.795	5.601	5.417	5.242	5.076	4.917	4.767	4.623	4.496	4.355	4.231	4.111	3.998	3.889	3.784	3.685	3.589	3.498	3.410	3.326	2.951	2.643
7	6.728	6.472	6.230	6.002	5.786	5.582	5.389	5.206	5.033	4.868	4.712	4.564	4.423	4.288	4.160	4.039	3.922	3.812	3.706	3.605	3.161	2.902
8	7.652	7.325	7.020	6.733	6.463	6.210	5.971	5.747	5.535	5.335	5.146	4.968	4.799	4.639	4.487	4.344	4.207	4.078	3.954	3.837	3.329	2.925
9	8.566	8.162	7.786	7.435	7.108	6.902	6.515	6.247	5.995	5.759	5.537	5.328	5.132	4.946	4.772	4.607	4.451	4.303	4.163	4.031	3.463	3.019
10	9.471	8.983	8.530	8.111	7.722	7.360	7.024	6.710	6.418	6.145	5.889	5.650	5.426	5.216	5.019	4.833	4.659	4.494	4.339	4.192	3.571	3.092
11	10.368	9.987	9.253	8.760	8.306	7.887	7.499	7.139	6.905	6.495	6.207	5.938	5.687	5.453	5.234	5.029	4.836	4.656	4.486	4.327	3.656	3.147
12	11.255	10.575	9.954	9.385	8.863	8.384	7.943	7.536	7.161	6.814	6.492	6.194	5.198	5.660	5.421	5.197	4.988	4.793	4.611	4.439	3.725	3.190
13	12.134	11.348	10.635	9.986	9.394	8.853	8.358	7.904	7.487	7.103	6.750	6.424	6.122	5.842	5.583	5.342	5.118	4.910	4.715	4.533	3.790	3.223
14	13.004	12.106	11.296	10.563	9.899	9.295	8.745	8.244	7.786	7.367	6.982	6.628	6.302	6.002	5.724	5.468	5.229	5.008	4.802	4.611	3.824	3.249
15	13.865	12.849	11.966	11.118	10.380	9.712	9.108	8.559	8.061	7.606	7.191	6.811	6.462	6.142	5.847	5.575	5.324	5.092	4.876	4.675	3.859	3.268
16	14.718	13.578	12.561	11.652	10.838	10.106	9.447	8.851	8.313	7.824	7.379	6.974	6.604	6.265	5.954	5.668	5.405	5.162	4.438	4.730	3.887	3.283
17	15.562	14.292	13.166	12.166	11.274	10.477	9.763	9.122	8.544	8.022	7.549	7.120	6.729	6.373	6.047	5.749	5.475	5.222	4.990	4.775	3.910	3.295
18	16.398	14.992	13.754	12.659	11.690	10.828	10.059	9.372	8.756	8.201	7.702	7.250	6.840	6.467	6.128	5.818	5.534	5.273	5.033	4.812	3.928	9.304
19	17.226	15.678	14.324	13.134	12.085	11.158	10.336	9.604	8.950	8.365	7.839	7.366	6.938	6.550	6.198	5.877	5.584	5.316	5.070	4.843	3.942	3.311
20	18.046	16.351	14.877	13.590	12.462	11.470	10.594	9.918	9.129	8.514	7.963	7.469	7.025	6.623	6.259	5.929	5.628	5.353	5.102	4.870	3.954	3.316

Period Discount Rate			Annuity Factors																			
	1%	2%	3%	4%	5%	6%	7%	8%	9%	10%	11%	12%	13%	14%	15%	16%	17%	18%	19%	20%	25%	30%
21	18.857	17.011	15.415	14.029	12.821	11.764	10.836	10.017	9.292	8.649	8.075	7.562	7.102	6.687	6.312	5.973	5.665	5.384	5.127	4.891	3.963	3.320
22	19.660	17.658	15.967	14.451	13.163	12.042	11.061	10.201	9.442	8.772	8.176	7.645	7.170	6.743	6.359	6.011	5.696	5.410	5.149	4.909	3.970	3.323
23	20.456	18.292	16.444	14.857	13.489	12.303	11.272	10.371	9.590	8.883	8.266	7.718	7.230	6.792	6.399	6.044	5.723	5.432	5.167	4.925	3.976	3.325
24	21.243	18.914	16.966	15.247	13.799	12.550	11.469	10.529	9.707	8.965	8.348	7.784	7.283	6.835	6.434	6.073	5.746	5.451	5.182	4.937	3.961	3.327
25	22.023	19.523	17.413	15.622	14.094	12.783	11.654	10.675	9.823	9.077	8.422	7.843	7.330	6.873	6.464	6.097	5.766	5.467	5.195	4.948	3.985	3.320
26	22.795	20.121	17.877	15.983	14.375	13.003	11.826	10.810	9.929	9.161	8.488	7.896	7.372	6.906	6.491	6.118	5.783	5.480	5.206	4.956	3.968	3.330
27	23.560	20.707	18.327	16.330	14.643	13.211	11.987	10.935	10.027	9.237	8.548	7.943	7.409	6.935	6.514	6.136	5.798	5.492	5.215	4.964	3.990	3.331
28	24.316	21.281	18.764	16.663	14.898	13.406	12.137	11.051	10.116	9.307	8.602	7.984	7.441	6.961	6.534	6.152	5.810	5.502	5.223	4.970	3.992	3.331
29	25.066	21.844	19.188	16.984	15.141	13.591	12.278	11.158	10.198	9.370	8.650	8.022	7.470	6.983	6.551	6.166	5.820	5.510	5.229	4.975	3.994	3.332
30	25.808	22.396	19.000	17.292	15.372	13.765	12.409	11.258	10.274	9.427	8.694	8.055	7.496	7.003	6.566	6.177	5.829	5.517	5.235	4.979	3.995	3.332
40	32.835	27.355	23.115	19.793	17.159	15.046	13.332	11.925	10.757	9.779	8.951	8.244	7.634	7.105	6.640	6.233	5.871	5.548	5.258	4.997	3.999	3.333
50	39.196	31.424	25.730	21.482	18.256	15.762	13.801	12.233	10.962	9.915	9.042	8.304	7.675	7.133	6.661	6.246	5.880	5.554	5.262	4.999	4.000	3.333

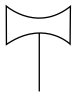
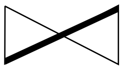
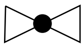
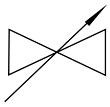
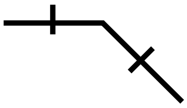
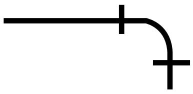
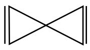
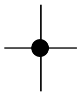
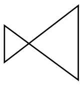
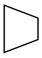
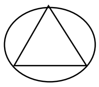
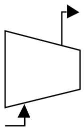
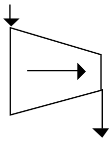
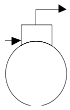
APPENDIX 4b: PRESENT VALUE FACTORS

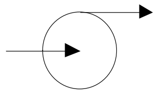
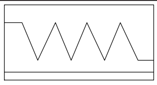
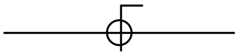
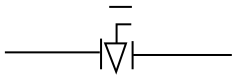
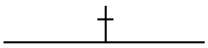
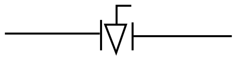
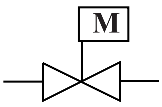
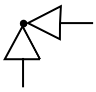
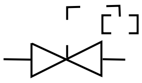
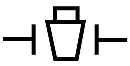
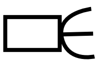
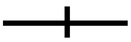
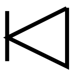
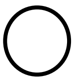
YEAR	1%	2%	3%	4%	5%	6%	7%	8%	9%	10%	11%	12%	13%	14%	15%	16%	17%	18%	19%	20%
1	0.990	0.980	0.971	0.962	0.952	0.943	0.936	0.926	0.914	0.909	0.901	0.898	0.885	0.877	0.870	0.862	0.856	0.847	0.840	0.833
2	0.880	0.801	0.940	0.926	0.907	0.889	0.876	0.807	0.842	0.820	0.812	0.787	0.705	0.769	0.750	0.640	0.701	0.710	0.700	0.684
3	0.871	0.842	0.910	0.908	0.904	0.840	0.810	0.784	0.772	0.701	0.701	0.712	0.685	0.675	0.690	0.641	0.624	0.608	0.690	0.678
4	0.861	0.824	0.888	0.866	0.828	0.762	0.768	0.765	0.708	0.688	0.660	0.636	0.613	0.662	0.672	0.552	0.534	0.516	0.400	0.482
5	0.851	0.826	0.866	0.822	0.764	0.747	0.718	0.681	0.650	0.691	0.586	0.567	0.543	0.518	0.497	0.476	0.456	0.487	0.418	0.402
6	0.842	0.838	0.836	0.790	0.711	0.705	0.688	0.660	0.596	0.564	0.535	0.507	0.480	0.456	0.432	0.410	0.390	0.370	0.362	0.335
7	0.833	0.871	0.815	0.760	0.677	0.665	0.623	0.588	0.547	0.513	0.482	0.452	0.425	0.400	0.376	0.354	0.386	0.314	0.296	0.278
8	0.823	0.860	0.789	0.701	0.646	0.627	0.602	0.540	0.502	0.267	0.404	0.404	0.376	0.351	0.327	0.250	0.265	0.260	0.249	0.209
9	0.814	0.837	0.768	0.708	0.614	0.562	0.644	0.500	0.460	0.424	0.391	0.361	0.388	0.308	0.284	0.238	0.243	0.226	0.200	0.194
10	0.805	0.820	0.741	0.679	0.585	0.558	0.508	0.463	0.422	0.386	0.352	0.322	0.285	0.270	0.247	0.227	0.206	0.191	0.176	0.162
11	0.88	0.804	0.722	0.660	0.557	0.524	0.465	0.429	0.388	0.350	0.352	0.284	0.261	0.234	0.216	0.185	0.168	0.162	0.148	0.135
12	0.887	0.789	0.701	0.625	0.506	0.497	0.444	0.397	0.350	0.319	0.316	0.257	0.310	0.205	0.187	0.165	0.152	0.157	0.124	0.112
13	0.079	0.770	0.691	0.601	0.500	0.489	0.415	0.380	0.320	0.290	0.266	0.228	0.204	0.102	0.160	0.145	0.130	0.110	0.104	0.080
14	0.870	0.769	0.661	0.577	0.506	0.442	0.389	0.340	0.299	0.263	0.250	0.205	0.181	0.182	0.141	0.125	0.111	0.090	0.088	0.780
15	0.861	0.748	0.642	0.555	0.481	0.417	0.362	0.315	0.275	0.230	0.232	0.188	0.160	0.160	0.123	0.108	0.095	0.084	0.071	0.065
16	0.853	0.728	0.628	0.353	0.458	0.394	0.339	0.292	0.252	0.218	0.209	0.163	0.141	0.140	0.104	0.098	0.081	0.071	0.062	0.054
17	0.844	0.714	0.606	0.513	0.436	0.371	0.317	0.270	0.231	0.196	0.188	0.140	0.125	0.128	0.093	0.080	0.069	0.060	0.052	0.045
18	0.833	0.700	0.587	0.494	0.410	0.350	0.286	0.250	0.212	0.180	0.170	0.130	0.111	0.085	0.081	0.069	0.059	0.051	0.044	0.040
19	0.28	0.686	0.570	0.476	0.396	0.331	0.277	0.232	0.194	0.164	0.155	0.116	0.098	0.088	0.070	0.060	0.051	0.048	0.037	0.031
20	0.82	0.678	0.554	0.456	0.377	0.312	0.258	0.215	0.178	0.140	0.133	0.010	0.087	0.073	0.061	0.051	0.048	0.037	0.031	0.026

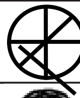
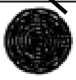
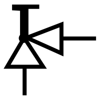
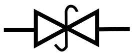
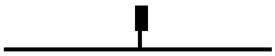
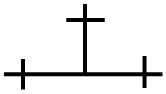
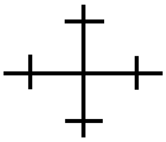
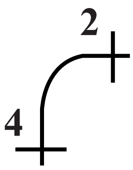
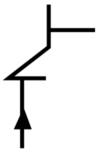
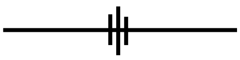
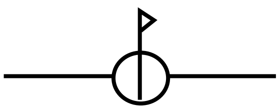
APPENDIX 5: RECOMMENDED WATER SUPPLY DRAWING SYMBOLS

APPENDIX 5a: Recommended Water Supply Drawing Symbols

APPENDIX 5a: Recommended Water Supply Drawing Symbols

NO.	*SYMBOL*	*DESCRIPTION*
1		water flow meter
2		valve
3		Globe valve
4		Gate valve
5		check valve
6		elbow 45^0
7		elbow 90^0
8		Flanged valve
9		crossing pipe, connected
10		reducing valve
11		reducer
12		pump
13		centrifugal pump
14		compressor /turbine
15		reciprocating pump

NO.	*SYMBOL*	*DESCRIPTION*
16		rotary pump
17		screw pump
18		Pipe guide
19		Butterfly
20		Plug Valve
21		Hose Bib
22		Plug Valve
23		Motor Operated valve
24		Angle gate Valve
25		Float Valve
26		Stop Cock
27		Cap
28		Joint
29		Flanged Reducer
30		Multi – Fit Coupling
31		Exixting Fire Hydrant

NO.	*SYMBOL*	*DESCRIPTION*
32		Proposed Fire Hydrant
33		Existing Isolating Valve
34		Proposed Isolating Valve
35		Angle globe Valve
36		Safety Valve
37		Water Hammer Arrestor
38		Tee
39		Cross
40		Expansion Joint
41		Reducing Elbow
42		Angle Check Valve
43		Union
44		Pressure Reducing valve

NO.	*SYMBOL*	*DESCRIPTION*
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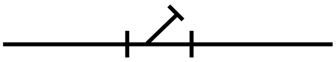
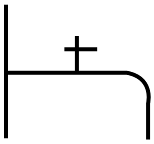
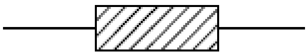
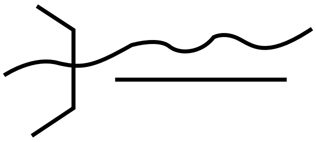
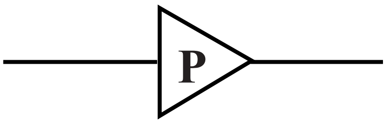
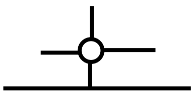
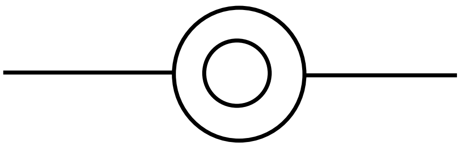
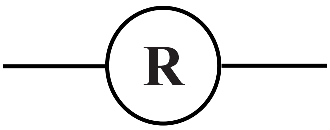
46		Wye
47		Meter
48		Tap
49		Treatment Works
50		Tap
51		Pumping Stations
52		Bore hole
53		Public Water Points
54		Break pressure tanks
55		Storage reservoir

APPENDIX 6: OTHER GRAPHS AND FORMULAE

APPENDIX 6a: A Moody chart for determination of friction factor.

APPENDIX 6b: Pipe Diameter Design

APPENDIX 6c: Hazen-Williams Coefficients and head loss formula

APPENDIX 6d: PVC and GI Friction Loss Tables

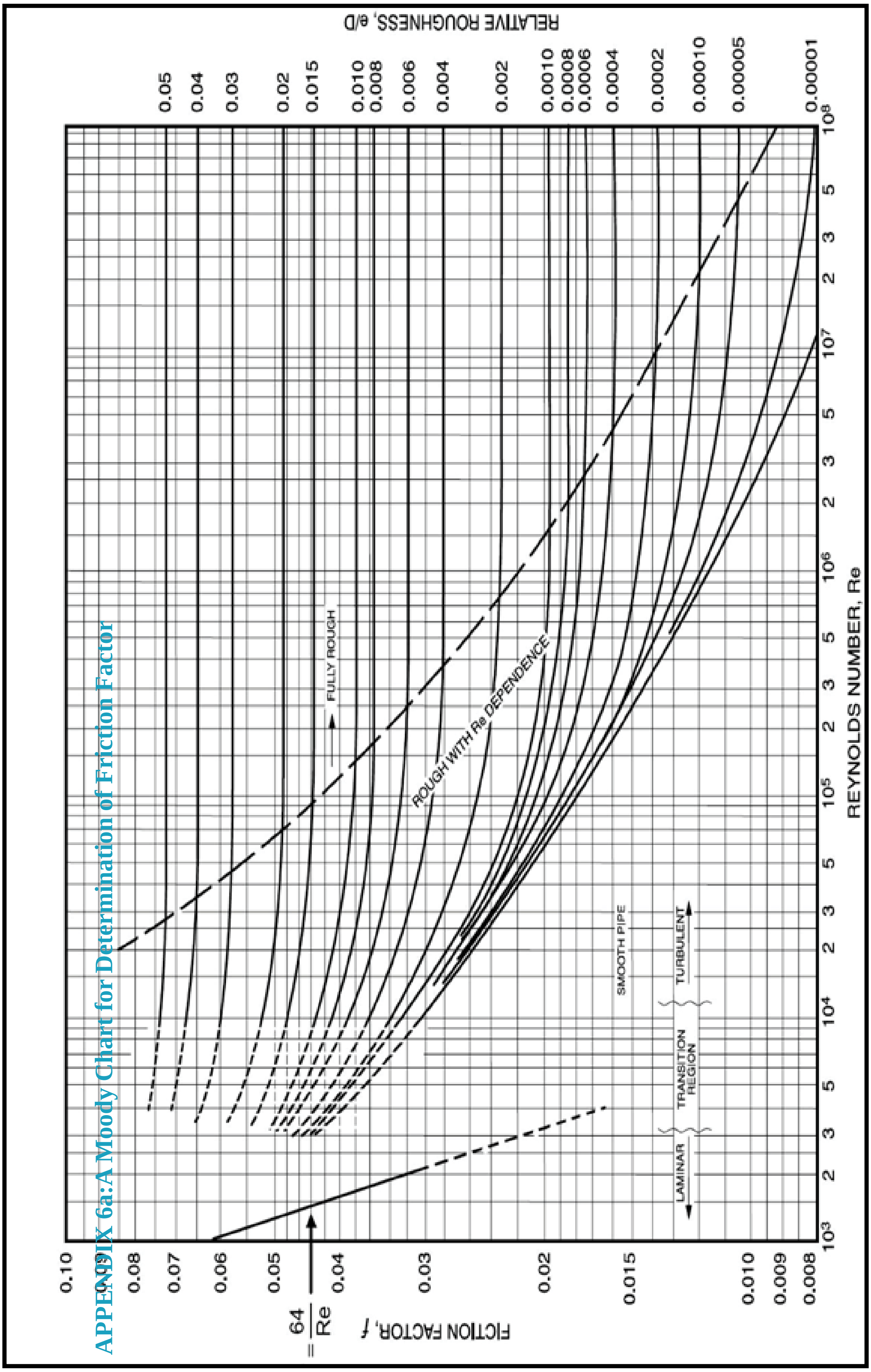
APPENDIX 6e: Sample Well Log

APPENDIX 6f: Sample Water Quality Analysis Report

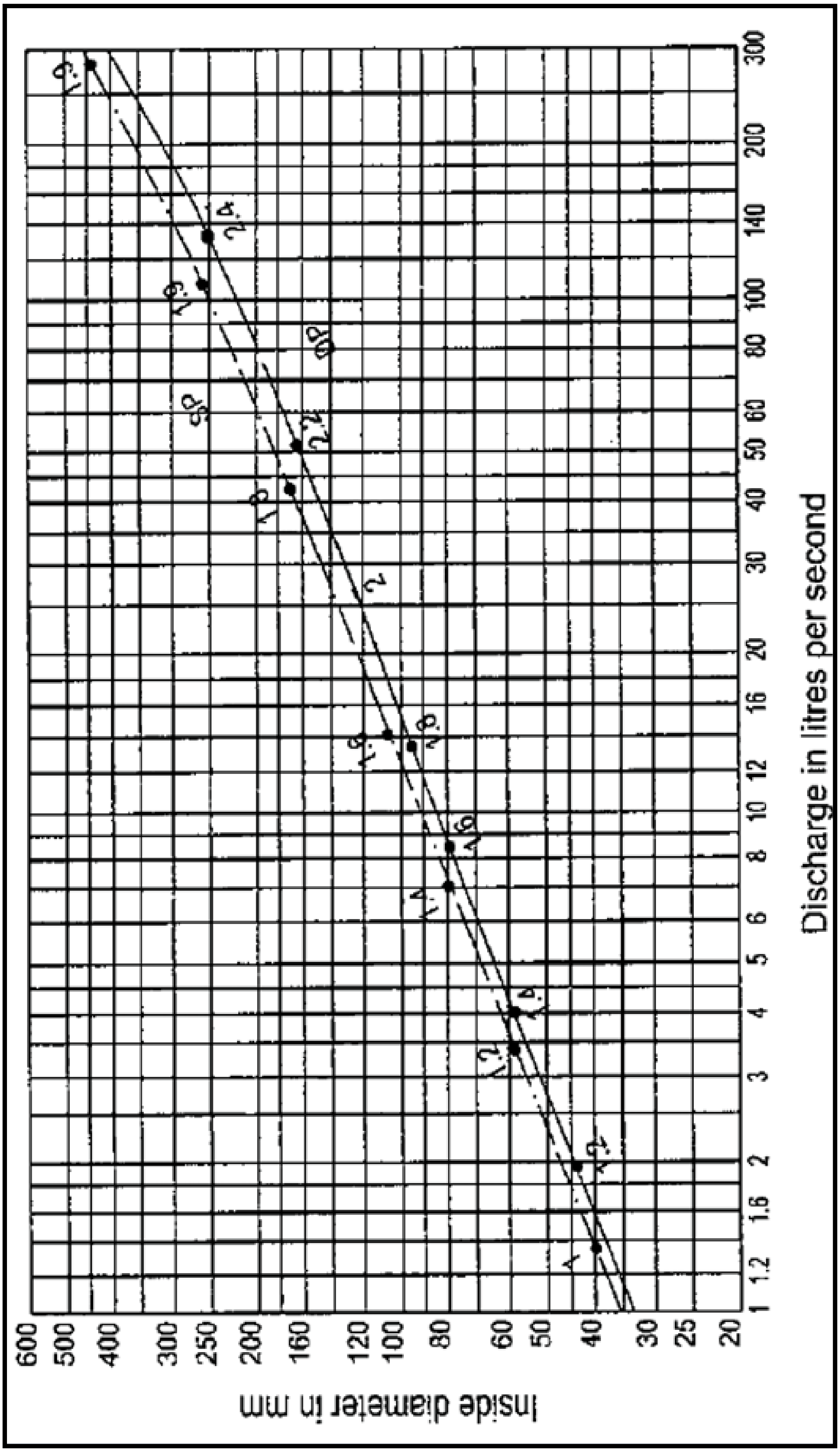
APPENDIX 6g:Time Draw Down graph on semi log paper

APPENDIX 6a:A Moody Chart for Determination of Friction Factor

APPENDIX 6a:A Moody Chart for Determination of Friction Factor



APPENDIX 6b: Pipe Diameter Design



APPENDIX 6c: Hazen-Williams Coefficients and Head Loss Formula

Typical *C* factors used in design, which take into account some increase in roughness as pipe ages are as follows (Hazen-Williams Coefficients, Engineering Tool Box)

Material	C Factor low	C Factor high
Asbestos-cement	140	140
Cast iron new	130	130
Cast iron 10 years	107	113
Cast iron 20 years	89	100
Cast iron 30 years	75	90
Cast iron 40 years	64	83
Cement-Mortar Lined Ductile Iron Pipe	140	140
Concrete	100	140
Copper	130	140
Steel	90	110
Galvanized iron	120	120
Polyethylene	140	140
Polyvinyl chloride (PVC)	150	150
Fibre-reinforced plastic (FRP)	150	150

General Hazen Williams formulae

$$V = kCR^{0.63}S^{0.54}$$

and exponentiating each side by 1/0.54 gives (rounding exponents to 2 decimals)

$$V^{1.85} = k^{1.85}\, C^{1.85}\, R^{1.17}\, S$$

Rearranging gives

$$S = \frac{V^{1.8}}{k^{1.8}\ C^{1.8}\ R^{1.7}}$$

The flow rate $Q = VA$, so

$$S = \frac{V^{1.8}\ A^{1.8}}{k^{1.8}\ C^{1.8}\ R^{1.7}\ A^{1.8}} = \frac{Q^{1.8}}{k^{1.8}\ C^{1.8}\ R^{1.7}\ A^{1.8}}$$

The hydraulic radius*R* (which is different from the geometric radius *r*) for a full pipe of geometric diameter d is d/4; the pipe's cross sectional area *A* is $\pi d^2/4$, so

$$S = \frac{4^{1.7}\ 4^{1.8}\ Q^{1.8}}{\pi^{1.8}\ k^{1.8}\ C^{1.8}\ d^{1.7}\ d^{3.0}} = \frac{4^{3.0}\ Q^{1.8}}{\pi^{1.8}\ k^{1.8}\ C^{1.8}\ d^{4.8}} = \frac{4^{3.0}\ Q^{1.8}}{\pi^{1.8}\ k^{1.8}\ C^{1.8}\ d^{4.8}} = \frac{7.916Q^{1.8}}{k^{1.8}\ C^{1.8}\ d^{4.8}}$$

When used to calculate the head loss with the International System of Units, the equation becomes:

$$h_f = \frac{10.67\, L\, Q^{1.85}}{C^{1.85}\, d^{4.87}}$$

http://en.wikipedia.org/wiki/Hazen-Williams_equation

Where h_f = head loss over a length of pipe, m (head pressure), L = length of pipe, m (meters),

Q = volumetric flow rate, m^3/s (cubic meters per second) and d = inside pipe diameter, m (meters).

HEAD LOSS IN METRES PER 100M FOR DIFFERENT CLASSES OF PVC AND GI PIPES

FLOW (M³/ h)	3/4"			1"			1 1/4"			1 1/2"				2"				2 1/2"				3"				
	PVC		**GI**	**PVC**		**GI**	**PVC**		**GI**	**PVC**			**GI**	**PVC**			**GI**	**PVC**			**GI**	**PVC**				**GI**
1	D	E		D	E		D	E		C	D	E		C	D	E		C	D	E		B	C	D	E	
1.5	2.4	3.2	8.0			1.9			1.5																	
2	5.0	6.8	17.8	1.6	2.2	4.2			2.6																	
2.5	9.6	11.5	31.9	2.8	3.8	7.5	1.3	1.8	3.9				1.5													
3	12.9	17.4	49.8	4.2	5.7	11.6	2.0	2.7	5.7				2.3													
3.5	19.0	25.8	71.7	6.2	8.4	17.7	2.7	3.7	8.0	1.2	1.4	1.9	3.3													
4				8.5	11.6	25.2	3.5	4.7	10.3	1.5	1.8	2.4	4.3				1.0									
5				10.8	14.8	32.7	4.9	6.7	13.3	2.2	2.6	3.5	6.3				1.5									
6				13.3	21.3	50.5	6.9	9.3	21.7	3.1	3.6	4.9	9.0	1.0	1.2	1.6	2.1									
7				21.6	29.6	72.7	9.2	12.5	29.3	4.1	4.7	6.5	12.3	1.4	1.6	2.2	2.9									
8				28.8	41.8	98.9	11.5	15.6	38.8	5.1	5.9	8.1	15.6	1.7	2.0	2.7	3.7				1.2					
9							14.4	19.6	49.1	6.4	7.5	10.2	20.0	2.1	2.5	3.4	4.7				1.6					
10							17.5	23.8	60.7	7.8	9.0	12.4	24.6	2.6	3.0	4.1	5.8				1.9					
12								33.3	87.4	10.8	12.6	17.3	35.3	3.6	4.2	5.8	8.4	1.2	1.4	1.9	2.7					1.1
14										14.3	13.9	22.9	48.3	4.8	5.6	7.6	11.5	1.6	1.8	2.5	3.7					1.5
16										18.3	25.9	29.3	63.0	6.1	7.2	9.8	16.2	2.0	2.3	3.2	5.0					2.0
18														7.6	8.9	14.4	20.9	2.5	2.9	4.0	6.3	1.0	1.0	1.3	1.8	2.5
20														9.2	10.8	18.7	25.6	3.0	3.5	4.9	7.6	1.2	1.4	1.6	2.2	3.1
					2"				2 1/2"					3"					4"					6"		
					PVC			GI	PVC			GI	PVC				GI	PVC				GI	PVC			GI
					C	D	E		C	D	E		B	C	D	E		B	C	D	E		B	C	D	
20					9.2	10.8	18.7	25.6	3.0	3.5	4.9	7.6	1.2	1.4	1.6	2.2	3.1									
25					13.9	16.2	28.7	37.3	4.5	5.3	7.4	12.0	1.8	2.1	2.4	3.3	4.8					1.1				

30									6.4	7.5	10.4	17.2	2.5	2.9	3.4	4.7	6.9					1.7				
35									8.5	9.9	13.8	23.2	3.4	3.8	4.5	6.2	9.3	1.0	1.1	1.3	1.8	2.3				
40									10.9	12.5	17.7	30.3	4.3	4.9	5.8	8.0	12.2	1.2	1.4	1.7	2.3	2.9				
45									13.6	16.3	22.0	38.2	5.4	6.1	7.2	9.9	15.4	1.5	1.8	2.1	2.9	3.7				
50														7.5	9.2	12.0	19.0	1.9	2.2	2.5	3.5	4.6				
60														10.7	13.1	17.2	27.6	2.7	3.1	3.6	5.0	6.5				
70														14.3	17.8	23.0	37.3	3.6	4.1	4.8	6.7	8.8				1.2
80														17.8	23.3	28.8	48.7	4.5	5.2	6.1	8.4	11.5				1.5
90																		5.6	6.5	7.6	10.9	14.6				1.9
100																		6.8	7.9	9.2	13.5	18.0	1.0	1.2	1.4	2.4
120																		9.5	11.0	13.6	19.5	25.9	1.4	1.7	1.9	3.4
140																							1.9	2.2	2.6	4.7
160																							2.4	2.8	3.3	6.3
180																							3.0	3.5	4.1	7.9
200																							3.6	4.2	5.0	9.5
225																							4.5	5.3	6.2	11.4
250																							5.5	6.4	7.8	15.0

Appendix 6e: Sample Well Log

P o box 36164 Kampala Uganda

LOG FOR DEEP WELL

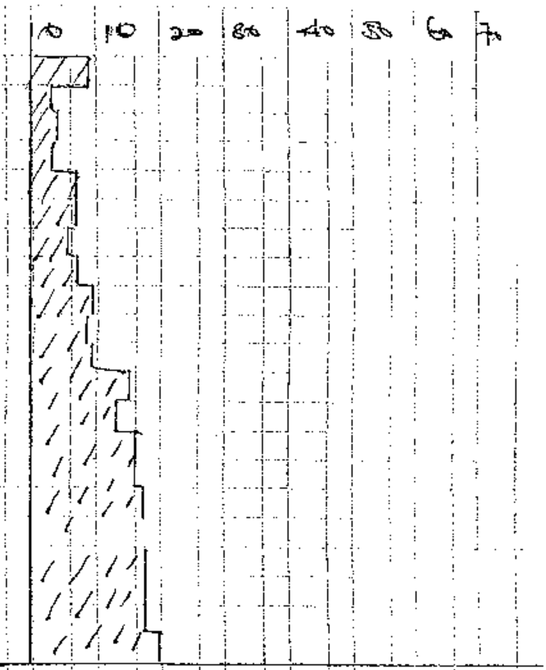
Village KACHERI HEALTH CENTRE	Village code	VES NO:
		Well No 1. DWD No.
Parish: KACHERI	Grid east 0602837	Well No. 2 34352
Sub County: KACHERI	Grid North 0350040	Distance from previous site 35Km
County JIE	Altitude ~~24/5/2011~~	Name of contractor GALAXY AGRC TECH'U' LIMITE
District KOTIDO	Date started 24/5/2011	Name of Driller KISITORO
Source Name	Date completed 24/5/2011.	

Date	Formation Log From (m)	To (m)	Litho logy, Supervisors description. (colour, hardness, moisture, grainsize, texture, Weathering, fractures, depth water struck..)	Problems, observations downtime and reason, e.t.c.
	0.00	5.18	Top soil black, grey	Table height is
			granites (grey dust)	1.00M
	5.18	9.78	Brownish fractured granites	Drilling started b.
			(brownish dust)	8" ODTH from 0.00M
	9.78	14.38	Brownish fractured granites	to 37.38M
			(slight dust)	No collapsion for
	14.38	18.98	Grey brownish fractured granites	Rods tripped out a
	18.98	23.58	Dark brownish fractured granites	6" ODTH inserted t
			(moist)	Counter sink the
	23.58	28.18	Grey brownish granites (moist)	hole further up to
	28.18	32.78	Grey brownish granites (moist)	92.3M
	32.78	37.38	Grey brownish granites (moist)	1st H_2O strike at 17.9m
	37.1	41.7	Grey brownish granites (moist)	2nd H_2O strike at 53.4
	41.7	46.3	Grey brownish granites (moist)	3rd H_2O strike at 71.2M
	46.3	50.9	Grey brownish granites (moist)	
	50.9	55.5	Grey granites 'Constant H_2O'	$Q = 6.6 m^3/hr$
	55.5	60.01	Grey granites 'constant H_2O'	
	60.01	64.7	Grey granites 'Constant H_2O'	
	64.7	69.3	Fresh grey granites	
	69.3	73.9	Fresh grey granites	
	73.9	78.5	Fresh grey granites	
	78.5	83.1	Fresh grey granites	
	83.1	87.7	Fresh grey granites	
	87.7	92.3	Fresh grey granites	

Checked by: ....

Supervisor

[illegible] om (m)	To (m)	[illegible]	Observation	Penetration ... Diameter mm	[illegible] (m)
0.0	5.18	09	Black	8" ODTH	
5.18	9.78	02	Brownish	(200MM)	
9.78	14.38	03	Brownish		
14.38	18.98	02	Brownish		
18.98	23.58	07	Brownish		
23.58	28.18	07	Brownish		
28.18	32.78	06	Brownish		
32.78	37.38	07	Brownish		
37.1	41.7	09	Brownish		
41.7	46.3	08	Brownish		
46.3	50.9	09	Brownish		
50.9	55.5	14	Grey		
55.5	60.01	12	Grey		
60.01	64.7	15	Grey		
64.7	69.3	15	Fresh		
69.3	73.8	16	Fresh	6" ODTH	
~~73.8~~	—	—	—	(150MM)	
73.9	78.5	16	Fresh		
78.5	83.1	16	Fresh		
83.1	87.7	16	Fresh		
87.7	92.3	19	Fresh		92.3



Appendix 6f: Sample Water Quality Analysis Report

NATIONAL WATER AND SEWERAGE CORPORATION

CENTRAL LABORATORY - BUGOLOBI.
P.O.BOX 7053 KAMPALA.
Tel: 257548, 341144, Fax: 256 41 255441
E-mail: waterquality@nwsc.co.ug

CERTIFICATE OF ANALYSIS

CLIENT: Galaxy Agro-Tec (U) Limited

Serial No: 2011/371-3

Sample Source: Borehole Water, Kotido District

Sampled by: Client

Date Sample Received: 10-06-2011

Date of Report: 15-06-2011

Table of Analytical Results

Parameters	Units	Village: Kotido S.S.S Parish: Kanawat S/County: Kotido County: Jie No: DWD 34347	National Standards for potable water. (Maximum Permissible)
WS Sample Nr.		K1633/11	
pH	--	7.27	6.5 – 8.5
Electrical Conductivity	µS/cm	1257	2500
Colour: apparent	PtCo	3	15
Turbidity	NTU	0.4	10.0
Total Dissolved Solids	mg/L	630	1200
Total Suspended Solids	mg/L	0	0.0
Alkalinity: total as $CaCO_3$	mg/L	430	500
Hardness: total as $CaCO_3$	mg/L	270	500
Calcium: Ca^{2+}	mg/L	60.0	75
Magnesium: Mg^{2+}	mg/L	28.8	50
Bi-Carbonate: as $CaCO_3$	mg/L	430	500
Chloride: Cl^-	mg/L	3.86	500
Fluoride: F^-	mg/L	0.13	1.5
Iron: total	mg/L	0.02	1.0
Sulphate: SO_4^{2-}	mg/L	11	200
Nitrate – N	mg/L	0.31	5.0

Remarks

The sample showed satisfactory physio-chemical characteristics of the source, which was commensurate with the National Standards for potable water quality. The source may be used for domestic & livestock water-supply.

NATIONAL WATER AND SEWERAGE CORPORATION
15 JUN 2011
THE CENTRAL LABORATORY

Herbert Wataga
SENIOR QC OFFICER

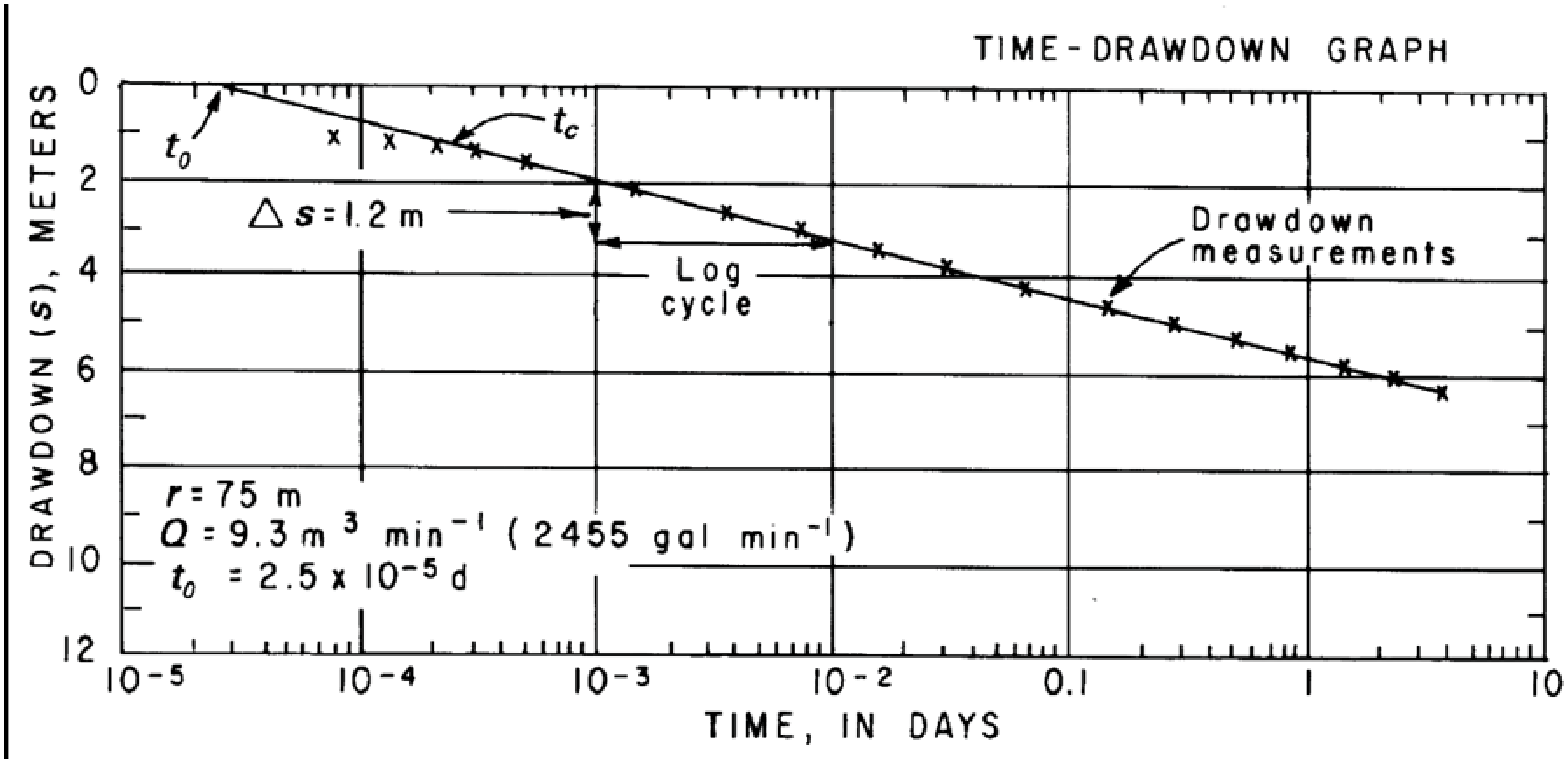
Lance E. Okwerede
For QUALITY CONTROL MANAGER

NB: The NWSC certificate of analysis by no means constitutes a permit to any person or undertaking to conduct business

Note:

Parameters maybe added or removed as per the requiremnt of the user

Appendix 6g:Time Draw Down graph on Semi Log Paper



Source: Basic groundwater hydrology by Ralph C. Heath, U.S. Geological Survey Water-supply Paper 2220, 1987, 84 pp.

Free download as pdf-file at: http://pubs.er.usgs.gov/pubs/wsp/wsp2220

Description of Parameters on the Graph

The Jacob method is applicable only to the zone in which steady-shape conditions prevail or to the entire cone only after steady-state conditions have developed. This time t_c in minutes for which steady-shape conditions are achieved can be calculated as.

$$t_c = \frac{7200\, r^2 S}{T} \tag{1}$$

where, r is the distance from the pumping well, meters, S is the estimated storage coefficient (dimensionless), and T is the estimated transmissivity in square meters per day.

After steady-shape conditions have developed, the drawdowns at an observation well begin to fall along a straight line on semilogarithmic graph paper as shown in the figure above. Before that time, the drawdowns plot below the extension of the straight line . When a time-drawdown graph is prepared, drawdowns are plotted on the vertical (arithmetic) axis versus time on the horizontal (logarithmic) axis.

The slope of the straight line is proportional to the pumping rate and to the transmissivity. Jacob derived the following equations for determination of transmissivity (T) and storage coefficient (C) from the time-drawdown graphs:

$$T = \frac{2.3Q}{4\pi \Delta s} \tag{2}$$

$$S = \frac{2.25T\, t_o}{r^2} \tag{3}$$

where Q is the pumping rate, Δs is the drawdown across one log cycle, t_o is the time at the point where the straight line intersects the zero-drawdown line, and r is the distance fromthe pumping well to the observation well.

Equations (2) and (3) are in consistent units,thus, if Q is incubic meters per day and s is in meters, T is in square metersper day. S is dimensionless, so that, in equation (3), if T is insquare meters per day, then r must be in meters and t_o must be in days. The reader is reffered toBasic groundwater hydrology by Ralph C. Heath for indepth discussion and calculation of Transmisivity, Storability and hydraulic conductivity.